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AMERICAN CONCRETE INSTITUTE

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PURPOSE OF THE INSTITUTE

It is the purpose of the Institute, by means of conventions and publications, to increase and disseminate knowledge in regard to the use of cement, concrete and allied products; to conduct research into their properties and uses; and to formulate recommended practice and standard specifications.

Its objects are purely educational.

JOURNAL
OF
AMERICAN CONCRETE INSTITUTE

FOUNDED 1905, AND INCORPORATED 1906, AS
NATIONAL ASSOCIATION OF CEMENT USERS
CHARTER AMENDED JULY 2, 1913

The Institute is not responsible, as a body, for the statements and opinions advanced in its publications

VOL. III JULY, 1915 NO. 7

SERVICE.

The members of the Institute have shown a most gratifying proof of their interest in its work by the promptness with which the annual dues are being paid. A delay in sending out the bills until recently has made it necessary for your directors to exercise special care in handling the publication of the JOURNAL, in order to avoid any hazardous strain on the credit of the organization. The JOURNAL is now being brought up to date rapidly, and your directors will be much helped in this work by the receipt of dues from those who have not already sent their annual remittances.

A feature of the Institute work which interests me even more at this time, is the addition to our membership of men who belong in our ranks, but have not yet joined. My own experience with a few acquaintances will prove to be typical of the experience of everybody, I am sure, who will try the method I followed. These acquaintances were men holding much the same position I occupy. It was merely necessary to tell them that the Institute was doing good work for the industry in which we were engaged, to secure their applications for membership. A personal talk or a telephone conversation is usually better than letters. Members of the Institute are necessarily men of good standing among their acquaintances. As such they have personal influence, and it is a small matter for each to use this influence forcefully for a few minutes each week to bring at least one new member into our organization.

The new members we most need are those who will be willing to contribute a little time to our work. Your directors can only accomplish the best results when all members will send to the Institute the suggestions and the criticisms they make in casual talks with each other. Otherwise your directors are left in lonely isolation, and they may miss an opportunity to render the assistance to you for which the Institute was organized and they were elected. They cannot read your minds, but they can read your letter of advice and criticism. The more they receive, the better they can meet your wishes. As the Institute's work develops, the individual members will be called upon for assistance from their experience and knowledge, and it is for this reason I have emphasized the value of new members who are good team workers.

LEONARD C. WASON.

REPORT OF COMMITTEE ON CONCRETE AND REINFORCED CONCRETE.

This JOURNAL contains the report of the Committee on Reinforced Concrete and Building Laws, presenting the full data of the column tests made at Pittsburgh, Pa., with the co-operation of the U. S. Bureau of Standards and the Engineering Experiment Station of the University of Illinois.

This is the most important series of tests of reinforced concrete columns of large size that has ever been made, and the data obtained, together with that obtained from tests of floor systems of reinforced concrete previously made and reported by this Committee, furnish valuable information as to the correctness of the present practice in the design of reinforced concrete buildings.

The Committee has been increased in membership and is making a further study of the results of these tests; later it will submit, in accordance with the resolution adopted at the last Annual Convention, a supplementary report in which additional conclusions will be given. This Convention also authorized the publication of the complete records of these tests, and it is hoped that additional conclusions will result from a discussion of the reports, which will be welcomed in the JOURNAL.

COMMITTEE ON PUBLICATIONS AND PAPERS.

Institute Notes.

Standards Adopted by the Institute.

1 Standard Specifications for Cement with Appendix on Standard Methods of Testing and Chemical Analysis, 23 pp.

2. Standard Specifications for Portland Cement Sidewalks, 6 pp.

4. Standard Building Regulations for the Use of Reinforced Concrete, 13 pp.

5. Standard Specifications for One Course Concrete Highway, 10 pp.

6. Standard Specifications for Portland Cement Curb and Gutter, 7 pp.

7. Standard Recommended Practice for the Use of Reinforced Concrete, 21 pp.

8. Standard Specifications for Scrubbed Concrete Surfaces, 3 pp.

9. Standard Recommended Practice for Concrete Drain Tile, 3 pp.

10. Standard Recommended Practice for Concrete Architectural Stone, Building Block and Brick, 4 pp.

11. Standard Specifications for Concrete Architectural Stone, Building Block and Brick, 3 pp.

12. Standard Building Regulations for the Use of Concrete Architectural Stone, Building Block and Brick, 3 pp.

13. Standard Specifications for Plain Concrete Floors, 6 pp.

14. Standard Specifications for Reinforced Concrete Floors, 5 pp.

15. Standard Specifications for Portland Cement Stucco on Metal Lath, Brick, Tile or Concrete Block, 9 pp.

16. Standard Methods for the Measurement of Concrete Work, 7 pp.

17. Standard Specifications for One Course Concrete Street Pavement, 10 pp.

18. Standard Specifications for Two Course Concrete Street Pavement, 11 pp.

19. Standard Specifications for One Course Concrete Alley Pavement.

The above Standards can be obtained at the price of 25 cents each or in lots of ten or more at 20 cents each. Special price to Members 15 cents each or 10 cents each in lots of ten.

Reduced Price for Back Volumes.

Members of the Institute can obtain copies of back publications at the following special prices:

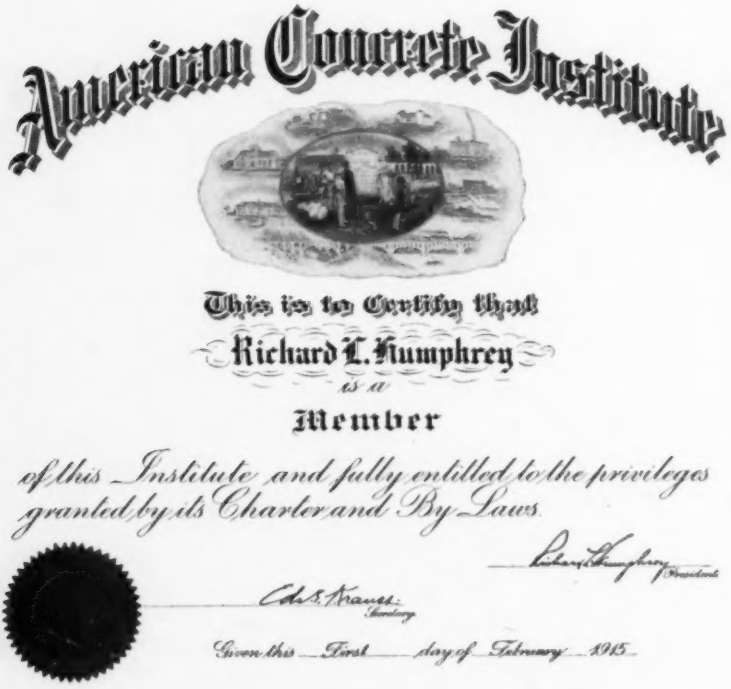
Vol. I, 50 cents; Vols. II to VI, $1.50 each; all in paper binding. For cloth or half morocco binding there is an additional charge per volume of 50 cents or $1.00 respectively. Vol. VII is available only in cloth binding, $2.00, or half morocco, $2.50.

Membership Certificate.

A Certificate of Membership in this Institute will be furnished to any member in good standing, i. e., whose dues are paid up to date. These Certificates are printed from an engraved steel plate. The name of the member is engrossed and the Certificate will be signed by the President and Secretary. The Certificates are about 10 x 14 ins., and have sufficient margin to be framed 14 x 17 ins. They are engraved on parchment or parchment paper, both suitable for rolling or framing.

The charge for certificates is as follows:

Parchment	$2.50
Parchment Paper	1.50


American Concrete Institute
This is to Certify that
Richard L. Humphrey
is a
Member
of this Institute and fully entitled to the privileges granted by its Charter and By Laws.
Given this First day of February 1915

MEMBERSHIP CERTIFICATE.

Synopses of Recent Articles on Concrete.

Reviewed July 1 to 31, 1915.

The synopses given below are for the purpose of keeping the members of the Institute in touch with the leading articles of interest appearing in the many technical journals and are not complete reviews of these journals. The publications regularly reviewed appear below and the number given corresponds to the number following the title of the paper. The length of the article in pages, whether illustrated or not, is noted, and the publications reviewed in which it appears:

1. Canadian Engineer, Toronto, Ontario, Canada.
2. Cement, New York, N. Y.
3. Cement and Engineering News, Chicago, Ill.
4. Cement Era, Chicago, Ill.
5. Cement World, Chicago, Ill.
6. Concrete Age, Atlanta, Ga.
7. Concrete Era, Los Angeles, Cal.
8. Concrete-Cement Age, Detroit, Mich.
9. Concrete and Constructional Engineering, London, England.
10. Engineering and Contracting, Chicago, Ill.
11. Engineering News, New York, N. Y.
12. Engineering Record, New York, N. Y.
13. Good Roads Magazine, New York, N. Y.
14. Portland Cement, Kansas City, Mo.
15. Rock Products, Chicago, Ill.
16. Proceedings American Railway Engineering Association, Chicago, Ill.
17. Western Contractor, Kansas City, Mo.
18. Journal National Fire Protection Association, Boston, Mass.
19. Journal Western Society of Engineers, Chicago, Ill.
20. Proceedings American Society of Civil Engineers, New York, N. Y.
21. Proceedings American Society of Municipal Improvements, New York.
22. Proceedings American Society for Testing Materials, Philadelphia, Pa.
23. Proceedings Engineers' Club of Philadelphia, Philadelphia, Pa.
24. Proceedings Engineers' Society of Western Pennsylvania, Pittsburgh, Pa.
25. Professional Memoirs, Corps of Engineers, U. S. A., Washington, D. C.
26. Proceedings Institution of Civil Engineers, London, England.
27. Transactions, Canadian Society of Civil Engineers, Montreal, Canada.
28. Railway Age Gazette, Chicago, Ill.
29. Better Roads, Jamestown, Ohio.

MATERIALS

The Economic Side of Sand Testing, by Cloyd M. Chapman and Nathan C. Johnson. (13) June 12, 1915. 3 pp., ill. A series of three articles, of which the first emphasizes the practical nature of the subject and shows how such testing effected a large saving for one contractor and secured a better concrete.

Safe Concrete Demands Knowledge of Nature of Sands, by Cloyd M. Chapman and Nathan C. Johnson. (13) June 19, 1915. 3 pp., ill. Second of a series of articles discusses origin and structure of grain and the value of tests for detecting impurities which impair the strength of the finished concrete.

Quality of Concrete Controlled by Tests of Sand, by Cloyd M. Chapman and Nathan C. Johnson. (13) June 26, 1915. 3 pp., ill. Third and final article of series proposes check on materials by means of portable device which simplifies usual analysis routine and gives direct graphical records. Existing test methods are critically discussed.

Locomotive Cranes to Economize Storage. (16) June 7, 1915. 4 pp., ill.

Two New Sand and Gravel Plants. (4) June, 1915. 3 pp., ill. Modern plants just completed to handle immense deposits at Waukesha. Layout and description.

DESIGN AND TESTS

Design Methods in Concrete Construction, Columns and Footings, by Jerome Cochran. (8) June, 1915. 3 pp., ill. Mathematical treatise.

BUILDINGS

How to Erect Small Concrete Buildings. (3) June, 1915. 2 pp., ill.

Design and Construction of the Chicago Hebrew Institute. (11) June 23, 1915. 5 pp., ill. A reinforced concrete structure possessing unusual features. Showing details and plans.

Design and Construction of the Gantner-Mattern Company Knitting Mills at San Francisco, by E. F. Cykler. (8) June, 1915. 5 pp., ill. Details and description of six-story building 60 x 103 ft.

Field Methods in Concrete Construction Beam and Slab Forms, by Jerome Cochran. (8) June, 1915. 4 pp., ill.

Empire and India Houses, Kingsway, by A. Lakeman. (9) June, 1915. 10 pp., ill. The reinforced concrete features are the staircase and floors.

BRIDGES AND RAILROAD WORK

Notes on Design and Construction of Provincial Highway Bridges at Mestre, Italy, by Albert M. Wolf. (8) June, 1915. 2 pp., ill.

Concrete Bridges with Stone and Brick Facing, by C. E. Drayer. (12) June 24, 1915. 2 pp., ill.

RESERVOIRS, DAMS, CANALS, ETC.

Reinforced Concrete Structure at Austin, Texas, Replaces Dam which Failed. (13) May, 2 pp., ill. Part II, which describes the departures from customary hollow-dam design, and the crest and sluice gates.

Concrete Rings, Superimposed, Sunk to Form San Antonio Pump Pit, by Wm. Wren Hay. (13) June 12, 1915. 1 pp., ill. Water supply company's new high-duty turbine-driven centrifugal units are direct-connected to the service mains.

Austin's Hydroelectric Plant has Unusual Reinforced-Concrete Wheel Castings. (13) June 12, 1915. 2 pp., ill. Part III—Article on reinforced-concrete structure replacing noted Texas dam which failed fifteen years ago.

One Huge Single-Life Lock at Louisville Will Guard the Entrance to the Portland Canal. (13) June 26, 1915. 3 pp., ill. Walls are built to the full height of 53 ft. behind forms supported by a mounted structural-steel frame. Ledge or hard rock makes drilling arduous.

Construction Features of Power Development Work at Eugenia Falls, Ont. (10) June 2, 1915. 3 pp., ill.

Pittsburgh's New Reservoir. (16) June 22, 1915. 1 p., ill. Interesting features of construction.

Concreting the Spaulding Dam. (12) June 3, 1915. 3 pp., ill. Dam 305 ft. high, of concrete. The concrete was mixed in a plant high up on the side of the canyon and chuted into place.

Reconstruction of Austin, Texas, Masonry Dam, by Frank S. Taylor. (12) June 3, 1915. 5 pp., ill. The work comprises hollow reinforced-concrete section replacing length washed out.

New Rating Flumes for Current Meters, Bureau of Standards. (12) June 10, 1915. 1 p., ill. Methods of construction.

ROADS AND PAVEMENTS

Thin Concrete Base, Reinforced, May Save 50 Cents a Square Yard in Paving Costs. (13) May, 1915, ¾ p., ill. Prof. James S. Tucker, in discussing results of slab test at Oklahoma University, foresees marked economies from reduced foundation thickness.

The Concrete Pavements of Sioux City, Iowa, by T. H. Johnson. (12) June 10, 1915. 2 pp., ill. The Sioux City methods of building concrete pavements have been characterized as a radical departure from current methods and a violation of some of the prime essentials of supposedly good practice.

SEWERS, PIPE, TILE, ETC.

The Construction of the Sooke Gravity Flow Line at Victoria, B. C. (8) June, 1915. 2 pp., ill. The work consists of the installation of 27 miles of 42-in. reinforced concrete conduit, of which 26 miles is gravity flow line conduit laid on a grade of 1 ft per 1000. The remaining mile consists of six inverted siphons, varying in head from 40 to 100 ft.

Expansion Joints for Monolithic Concrete Pressure Pipe. (1) June 10, 1915. 1 p., ill. Showing design and arrangement of joints.

CONCRETE PRODUCTS

The New Delaware & Hudson Office Building at Albany, N. Y., by Marcus T. Reynolds. (8) June, 1915. 3 pp., ill. The trim of the handsome new building of the Delaware & Hudson R. R. office building at Albany, N. Y., shown in the accompanying illustrations, is of concrete stone.

MISCELLANEOUS

New Instrument Tests Sands Quickly in the Field. (13) June 26, 1915. 2 pp., ill.

Methods Used In Constructing a 100-Ft. Monolithic Concrete Silo Near Salona, Pa. (8) June, 1915. 2 pp., ill. The silo is 16 ft. in diameter and 108 ft. high, with a capacity of 610 tons.

Investigation of the Effects of Alkali on Concrete Drain Tile Near Lake Park, La., by Chas. E. Sims and Geo. P. Dieckman. (8) June, 1915. 3 pp. Includes report of chemist's investigations.

Reinforced-Concrete Floating Caissons for the Welland Ship Canal. (12) June, 24, 1915. 2 pp., ill. Details of design of construction.

Intercolonial Railway Pier 2 at Halifax Harbor, Nova Scotia, by A. F. Dyer. (12) June 10, 1915. 6 pp., ill. The reinforced-concrete pier on long unsupported concrete piles in an ocean harbor subjected to severe ice conditions. Special pile construction and driving.

United States Making Concrete Caissons for Sheboygan Breakwater. (4) June, 1915. 4 pp., ill. Details and construction of caissons 54 x 20 x 18 ft.

Some Elements of Business Success. (4) June, 1915. 1 p., ill. Finding the total cost of make and sell.

Reinforced Concrete Chimneys in Havana. (8) June, 1915. 1 p., ill. Design and construction features.

PART II.

PROCEEDINGS ELEVENTH ANNUAL CONVENTION.

Papers, Discussions and Reports.

MIXING, CURING AND PLACING CONCRETE WITH HIGH PRESSURE STEAM.

By Harold P. Brown.

The ideal concrete of the engineer and builder should be of uniform, monolithic structure with the strength and density of granite. It should be as waterproof as asphalt and as flameproof as fire clay. It should be mixed with only the proper amount of water, yet every particle of cement should receive perfect hydration. There should be no pause between the mixing and the placing; from the moment the water touches the cement until the concrete is poured into its final position, there should be continuous and thorough mixing. No crevices should be permitted between successive pourings, nor should there be any cracks or voids. The pourings should be followed by heavy tamping, but not a stroke should be allowed after the initial set. Perfect adhesion should be secured to all reinforcing steel and to earlier work.

At very reasonable cost, these ideals have been realized by the process and apparatus to be described in this paper. The resulting product weighs more than 170 lbs. to the cu. ft.; which is more than granite, while the same materials hand-tamped weigh but 137 lbs; it has 3 to 4 times the strength of average concrete and is to a certain extent crystalline and vitreous in structure; it has a fine appearance, a good color and will take a high polish.

In ordinary concrete work the amount of water is often left to the discretion of the man in charge of the mixer, who is apt to use an excess to secure easy mixing and pouring and quick cleaning. This excess of water gives abundant chance for voids and cracks. If the concrete is poured into barrows and wheeled to its place, the jarring during transit causes the stones to settle to the bottom; the tilting of the barrow does not restore the mix, since the stones are the last to fall. To secure easy flow, when concrete is hoisted in buckets and conveyed in chutes, too much water is often used; or if the amount is correct and a steeper

pitch is given to the chute, the water may rise to the surface, leaving the slower moving concrete to get a part of its initial set. The water, the mortar and the stones then fall in partly separated layers, and even the most thorough tamping cannot secure a proper mixture. The usual tamping is poorly done and does not produce solid, waterproof work, even when the best materials have been used. Many bad cracks between separate pourings

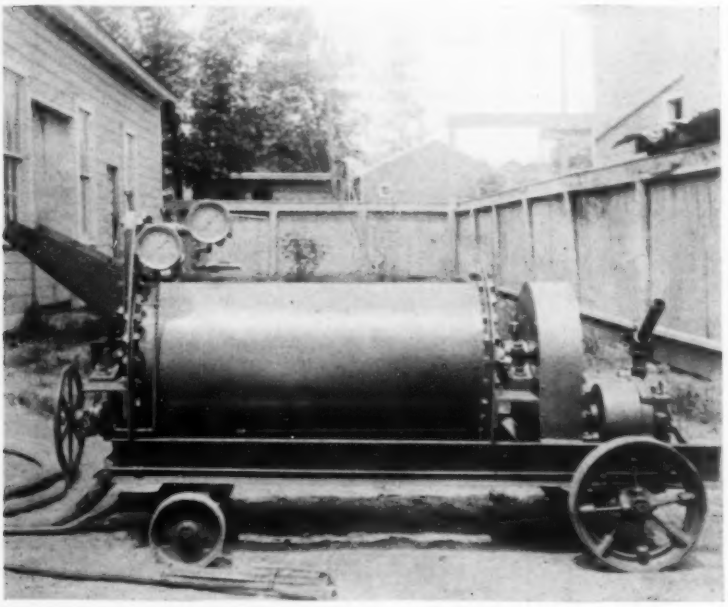


FIG. 1.—NO. 2 CONCRETE ATOMIZER USED ON LACKAWANNA R. R.

occur in nearly every job because the precautions necessary to obtain a fair bond are seldom taken.

It has long been known that the tensile and crushing strength of concrete blocks could be increased after their final set by treatment for several hours with steam under high pressure. The heat probably causes more or less opening of the pores and the pressure forces the moisture into the structure and completes the hydration of the cement. Mr. Rudolph J. Wig, of the United States Bureau of Standards, has made many careful experiments in this line, and

Mr. H. J. Force, chief chemist of the Lackawanna Railway, has reported similar effects upon cement-sand briquets treated in the autoclave. To prevent freezing of work it is common practice in winter to heat the concrete materials before mixing, and even this has produced a slight increase in strength.

My process consists in thoroughly mixing measured amounts of cement, sand, broken stone and water in a cylinder which is then filled with a hot gas or vapor under high pressure. Perfect hydration is thus secured and a most effective chemical action is set up which gives the product entirely new qualities. When the proper pressure and temperature have been reached and the curing process is completed, the outlet valve is opened and the plastic concrete is dropped into a super-heated stream of rapidly-moving gas or vapor. The wet particles are separated from each other and can be carried several hundred feet in a rubber-lined hose to the point of deposit which may be many feet above the mixer. The stream is projected through an expansible nozzle and strikes the work at high speed, thus tamping it and securing unusual density and strength. The blow jars out part of the water, and the high temperature and the sudden reduction of pressure cause rapid evaporation of the remaining moisture, so that quick setting is obtained. At first the stones or pebbles bound off, but when a layer of $\frac{1}{2}$ in. or more has been deposited, they imbed themselves and further tamp the work, making it adhere strongly to brick, stone, reinforcing steel or old concrete. A thin slab made in this way has an almost metallic ring and is waterproof at heavy pressure. When the proper amount of water is used very little concrete falls away, and layers over 1 ft. thick have been placed at one time on vertical walls. More water is used in the final coat which is thus made plastic enough for easy finishing. During each period of mixing a stream of super-heated steam is kept flowing at low pressure through the outlet to keep the hose and the destination of the new work warm and moist. With careful handling excellent work can be done in freezing weather.

The machine (Fig. 1) which for want of a better name I have called the "Concrete Atomizer," was rented by the Lackawanna Railway and used in repairing honeycombed work subject to tide-water at their Hoboken Terminal. Here compressed air

was used, but in the repair work along the road, steam was taken direct from the locomotive, thus saving the trouble and great expense of the cumbersome air compressor with its motor, receiver, regulator, water pump and tank. The operation of this atomizer was so satisfactory and its product of such high character that the engineer in charge has written a most unusual and appreciative letter on the subject, and in accordance with his advice the Road has recently purchased the apparatus for further extensive repairs and construction work.

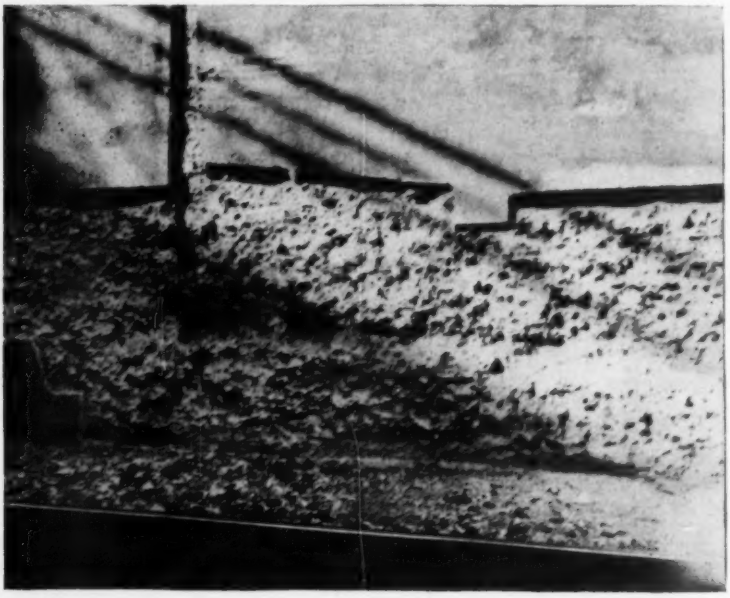


FIG. 2.—HOLE IN RETAINING WALL ON LACKAWANNA R. R. IN NEWARK, N. J.

Figs. 2 and 3 show the method of repairing a hole in a concrete retaining wall where the deepest portion was 15 in.

Fig. 4 shows the jet in operation. The hole referred to in Figs. 2 and 3 appears in the lower right corner. It can be seen that very little material has been wasted, for the ballast next to the wall has but little covering consisting of a small amount of sand and a few pebbles. These can be gathered up and used.

Fig. 5 is another view of the same work, showing the

manner in which the steam was obtained from the locomotive of a work train. The picture shows two men at the nozzle, but the duty of the extra man was merely to keep an eye on the watchman who announced the approach of trains.

FIG. 3.—HOLE SHOWN IN FIG. 2 FILLED WITH CONCRETE BEFORE SCREEDING.

One of my early experimental machines was used about 2 years ago on the New York Central and over 6000 sq. ft. of structural steel was covered with from 2 to 6 in. of concrete, applied with compressed air and with steam. All of this work has proved as hard as flint and not a single crack or fault has developed in it. This is true of all the work of my other larger machines.

When the atomizer was operated with air at another plant, a 60-h. p. boiler was used to supply steam for the compressor, and it consumed about 400 lbs. of Buckwheat coal per hour, using forced draught and producing about 1 cu. ft. of concrete per minute. When steam was used direct instead of air, the same amount of work was done with natural draught and less than 80 lbs. of coal per hour used. The cost of the fuel for mixing, curing and placing 1 cu. yd. of concrete with this little machine did not exceed $.10, and a boiler of 20 h. p. is large enough for it.

When steam is used the mixing is done at a pressure of 75 to 80 lbs. with sufficient superheat to prevent condensation, thus maintaining the proper amount of water in the concrete. By increasing the amount of superheat to the proper degree, the steam issuing from the nozzle is kept in gaseous form so that the work deposited can easily be seen by the operator. The superheater can be located in the stack of the boiler if the work is nearby, or over a salamander if distant. Or when the boiler is operated at 180 to 200 lbs. pressure, a reducing valve is placed close to the machine and set for 85 lbs. pressure supply to the mixer, thus obtaining the required superheat. While experimenting with the superheater, some interesting results were obtained with a lot of rejected cement whose neat briquets were like chalk. Many samples were mixed and the pressure and temperature gradually increased, until at last solid concrete was produced.

A special grade of rubber is used for the hose lining and this is worn but very slightly by the concrete, since wet cement to quite an extent lubricates the surface. I have many feet of hose which is still in good condition after severe service for more than a year, and it shows very little wear. Special couplings are used with no metal parts exposed to the concrete stream.

To avoid delay through clogging of the hose, an expansible rubber nozzle is used whose size of opening is controlled by a lever in the hand of the operator. In case a pebble blocks the flow, a slight movement of the lever opens the passage to the full diameter of the hose and the obstruction is blown out. The pressure at the nozzle is about 45 lbs. and the stream of mingled vapor and concrete shields the operator from rebounding pebbles, while wooden ferules on the nozzle levers protect his hands from the heat of the steam. A skillful operator can apply a thin,

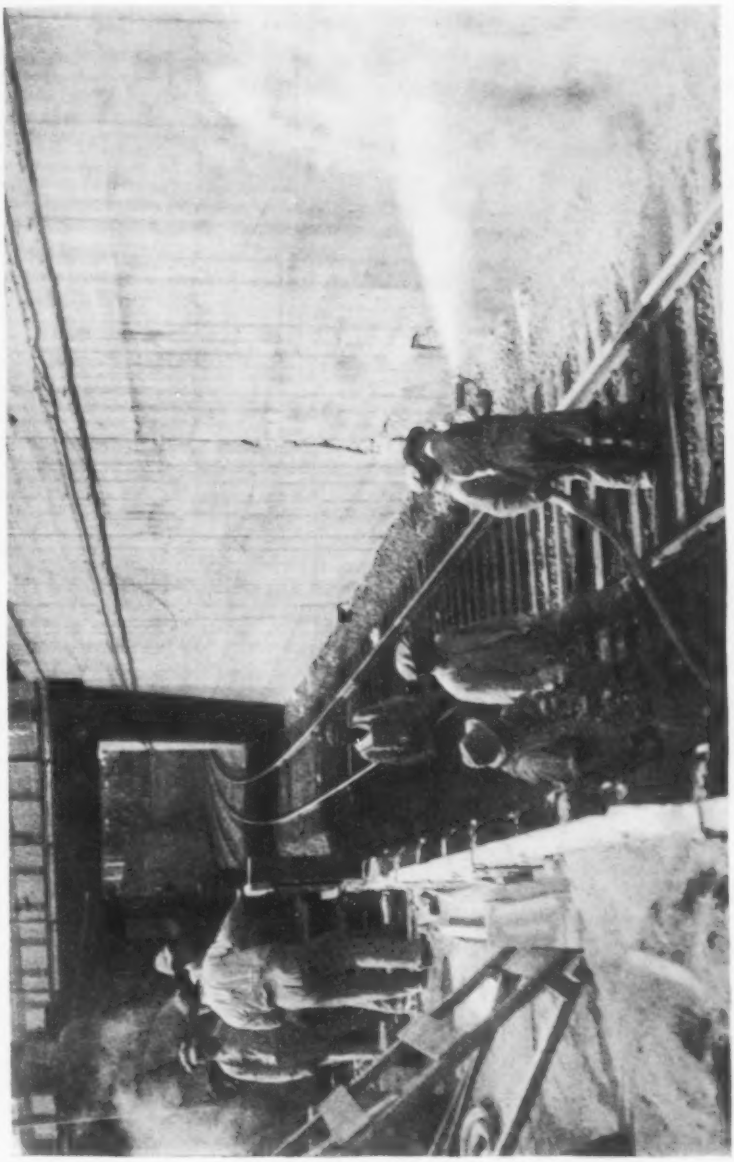


FIG. 4.—CONCRETE ATOMIZER OPERATED WITH STEAM ON THE LACKAWANNA R. R.

even layer of concrete almost as smooth as the scratch finish on plaster, or he can produce a surface lightly or heavily pebbled as may be desired. A glass finish can be given to the work by projecting it against an oiled sheet of planished metal.

The No. 2 Concrete Atomizer has a charging door either at one end or in a dome on the top. The driving shaft is square in section and carries a series of mixing paddles and scraping devices for keeping the surfaces clear. Ample clearance is given between the paddles and shell so that stones up to $\frac{3}{4}$ in. can be used. In operation the rapid motion of the rotating parts keeps the air filled with flying particles of concrete. These particles are driven through the outlet valve and discharged into the hose, which is $1\frac{3}{4}$ in. inside diameter. The speed of the engine automatically increases as the load grows less, so that the rate of discharge can be kept uniform. When the cylinder is nearly empty the tank pressure falls and its steam supply is shut off; the rest of the work is done by the expansion of the steam through the hose. The piston of the outlet valve is made to clear all material from its path when closing so that no dead passages are left in which concrete can set and block the pipes.

The No. 3 Concrete Atomizer is similar in construction and has a mixing tank 5 ft. long and 3 ft. inside diameter. It is designed to mix about 20 cu. ft. of loose materials with an output of 150 to 180 cu. ft. per hour; it will handle 1 in. stones. An engine of my own design is used with this machine, as the requirements of the service are unusual. The engine has a splash lubrication and is self-contained and powerful. It can be operated with compressed air when the machine is to be worked in tunnels or caissons where steam cannot be used. Even in this case the air can, if necessary, be superheated and the advantages secured of combined heat and pressure.

A prominent engineer is at present making careful records of the costs of curing and placing large masses of concrete with this machine and his report will be submitted in due time to this Institute with full details of the tensile and crushing strength of the product. It must be remembered that the only additional cost of the product, beyond the cost of ordinary power mixing, is the fuel required to heat the materials to about 350 deg. F. and to convey the concrete through the hose to its place in the work.

FIG. 5.—JETTING CONCRETE WITH STEAM ON THE LACKAWANNA R. R.

Operating with steam the cost of fuel and labor in mixing, treating and placing concrete with the small machine through 300 ft. of hose has been less than $.80 per cu. yd. With the larger machine the cost is about $.30. It is often objected that the 15 per cent to 20 per cent greater density obtained by the process causes a correspondingly greater cost per cu. ft. for materials. This is true, but as the strength of the product is multiplied by $2\frac{1}{2}$ to 3, the section can be reduced at least 25 per cent and the cost of materials put on the same basis as ordinary work with a greatly increased factor of safety. The same result can be obtained by using less cement in the mix. The Concrete Atomizer can be used to replace the ordinary mixer, engine, hoisting apparatus, tower, chutes and runways in all kinds of light construction.

The process opens up many new fields for the use of cement. The adhesion is so great that old cobblestone or granite block pavements can be resurfaced at low cost with a thin layer of dense, flinty concrete. The same treatment can be applied to worn-out macadam roads with good foundations. Earth irrigation ditches and canals can be strengthened and waterproofed. Frame buildings can be fireproofed by a thin self-supporting wall of reinforced concrete which can also take the weight of a light concrete roof. Slabs of concrete 1 in. thick with outer surfaces as smooth as planished steel can be easily made and used for forms in place of wood, becoming the outer part of the finished work, after the pouring is completed. This will reduce the cost of forms in certain classes of work. By the use of light reinforcement and thin removable metal sheets, hollow concrete house walls and floors can be quickly jetted into place at a lower cost than any other permanent construction. Such a building would be proof against fire, water, heat and cold and would greatly reduce the cost of living.

I believe that I have set before you the crude outlines of an entirely new structural art which will overcome all the present faults and disadvantages of concrete. I feel that you will soon become convinced that in making perfect concrete, heat and pressure are as necessary as in the manufacture of perfect brick. And when you reach that conclusion and act upon it, the age of Triumphant Concrete will begin its everlasting reign.

DISCUSSION.

MR. J. H. LYTLE.—I would ask Mr. Brown the size of the outlet hose. Mr. Lytle.

MR. H. P. BROWN.—It is ¾ in. inside diameter. Mr. Brown.

MR. L. J. MENSCH.—Did I understand Mr. Brown to say that with a 60 H. P. boiler he can throw out 1 cu. ft. a minute? Mr. Mensch.

MR. BROWN.—No, a 60 H. P. boiler was used at that particular time, and I gave you the amount of fuel that was required. That was to show the amount required when compressed air was used and when direct steam was used. A 30 H. P. boiler would have been sufficient for the purpose. With the 60 H. P. boiler, the fuel used is No. 3 buckwheat coal of a very low grade, so that it requires forced draft to burn it. That boiler running a big compresser, and then using the air for operating the machines, requires about 400 lbs. of coal per hour, and the output is about 1 cu. ft. per minute. When we used steam drive the fuel fell under natural draft to about 90 lbs. per hour with the same output. The larger machine will do about 40 per cent more work. Mr. Brown.

MR. MENSCH.—Why is it that you claim such great advantages for this material with this way of placing? Mr. Mensch.

MR. BROWN.—In the first place the pressure of the heat during the mixing forces out all air in the cement and aggregates. The heat causes the air to expand and the pressure forces the moisture into the interstices of the cement so that the hydration is perfect. There is also, I think, an increase of chemical activity. Some cements produce considerable heat in the process of hydration. If extra heat is supplied, greater density is obtained. This seems to me to be demonstrated by the poor cement, which would not make even a neat briquette, but with the pressure at about 90 lbs., gave dense and strong results. Mr. Brown.

MR. R. J. WIG.—I understand the steam is used under about 80 lbs. pressure. This had a temperature of about 315 or 318 deg. F. Did not this cause the cement to become quick setting? How long was the concrete kept in the atomizer? What is the basis for the statement that the concrete was 2½ times as strong Mr. Wig.

Mr. Wig. as that ordinarily obtained? Was there difficulty in getting a smooth or level surface? Was it necessary to finish the surface with a trowel?

Mr. Brown. MR. BROWN.—I used 90 lbs. in the mixing chamber and tried to have about 50 deg. of superheat. Of course as long as there is free water present superheat cannot be obtained; but after the mixing had taken place I seemed able to run the temperature up without any detrimental effects on the concrete; and there seemed to be no loss of strength. I have kept a batch running for 15 minutes. If I stopped with the machine full it would set quickly. As the stream of concrete flows from the machine it meets the superheated steam which joins it at the outlet valve, and carries it through the hose and deposits it on the work. The drop in pressure is regulated so that the nozzle pressure is about 45 lbs. I find each additional 50 ft. of hose requires about 5 lbs. more pressure of the stream of atomizer steam.

The question of smoothness of work depends altogether upon the skill of the nozzleman. He can apply on a vertical surface an almost even coating, so that the coating looks as though a man had applied it like a sand finish of ordinary plaster before the scratch coat comes on. A trowel ought not to be put on to this work—that is, when it is deep, 4 or 5 in. or more—it should not be touched until the final setting occurs; otherwise cracks may be caused. For a finish coat I put on about $\frac{1}{2}$ or $\frac{3}{4}$ in. and use a little more water, and drop off the bevel. That coat can be troweled without any trouble. Usually one nozzle will keep about two good masons troweling.

Mr. Wason. MR. L. C. WASON.—I would like to ask Mr. Brown about the unequal expansion due to applying this coating to steel; the steel cooling during the application of the cement.

Mr. Brown. MR. BROWN.—While the load is being received and mixed, a stream of steam is turned on to the steel that is to receive the work, so that it is heated. Of course if hot concrete were put on cold steel the latter would expand during the initial setting and that would produce cracks.

DESIGN AND CONSTRUCTION OF THE MASSACHUSETTS INSTITUTE OF TECHNOLOGY BUILDINGS.

By Sanford E. Thompson.*

The construction of the new buildings of the Massachusetts Institute of Technology represents one of the most comprehensive schemes, for an educational institution, that has been developed in this country. Standing upon an area of 50 acres, the buildings themselves cover an area of 3½ acres, and if placed in a straight line would extend some 2500 ft. in length. Except the stairways, which will be of structural steel, reinforced concrete is being used throughout, and about 40,000 cu. yds. of concrete and 3600 tons of steel reinforcement will be required.

The main subject of this paper is the structural features of the work, more especially the engineering studies and investigations that determined them. In connection with a general description of the layout of the work, however, the more interesting construction details are taken up.

Arrangement of Buildings.

The complete plan provides for over 30 buildings with 17 of the larger and more important grouped around a central court and so arranged and constructed as to form one continuous building. (See Fig. 1.)

More academic buildings will be erected in the rear of the present structure, on Vassar Street, when the money available and the number of students warrant. At present only the 13 buildings shown in full lines in Fig. 1 are being erected, but plans for all those shown dotted are well under way.

The buildings facing directly on the major and minor courts (see Fig. 4) and on the important streets are faced with Indiana

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limestone. On the secondary courts and other less prominent places pressed brick is used.

The buildings on the major court have 4 stories and those on the minor courts 3, but on these latter are 2 pavilions, one either side of the main entrance from the street, that run up to the fourth story.

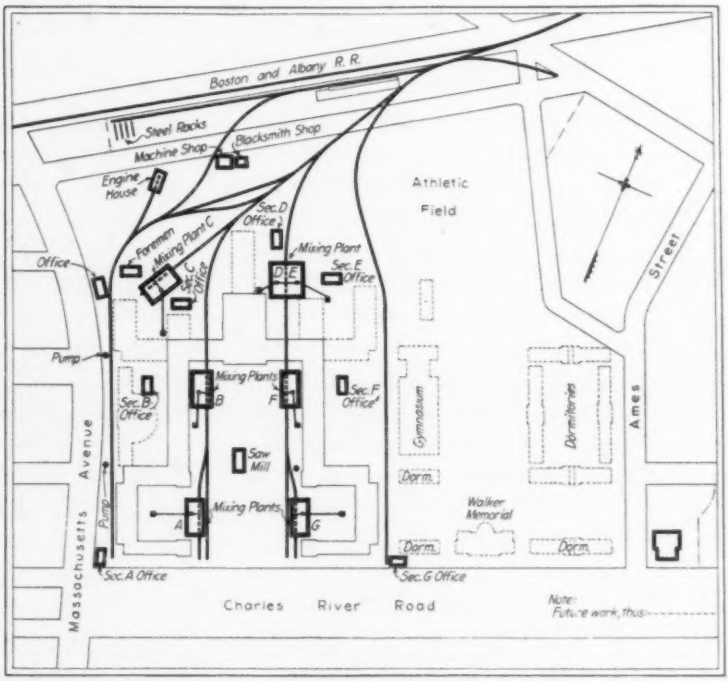

FIG. 1.—GENERAL LAYOUT OF PERMANENT BUILDINGS AND CONSTRUCTION PLANT.

CONSTRUCTION DETAILS.

The size of the job made it convenient for construction purposes to group the buildings into 7 sections, A to G inclusive. (See Fig. 1.) Each section has its own organization, with field office, mixing plant, etc.

Materials are carried direct by switch tracks from the Boston & Albany R. R. on trestles to each mixing plant. Sand and gravel are dumped under the trestle into pits holding respect-

ively 300 and 400 cu. yds. Cement is stored in houses, one for each plant. The $\frac{3}{4}$-yd. mixers are located in pits so that all materials are fed by gravity. The mixed concrete is poured into cars which are run up an inclined track to the hoist towers. These towers are located in central positions so that the concrete can be hoisted and distributed through chutes to the various sections. The mixed concrete is not, however, chuted directly to place, and therefore this source of danger of poor concrete is avoided. It was found, in order to handle the concrete by

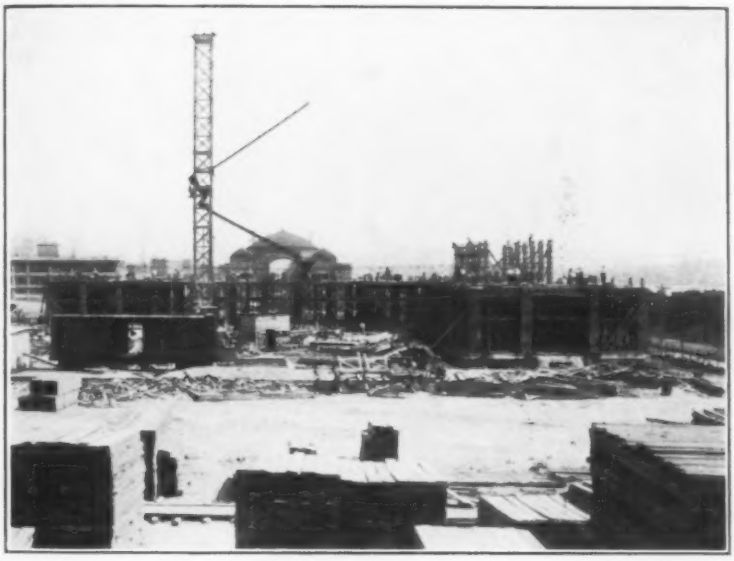

FIG. 2.—TYPICAL ARRANGEMENT OF TOWER, CHUTE AND HOPPER.

chutes and at the same time maintain the proper consistency, that it was necessary to limit the slope of the chutes to 30 deg. with the horizontal. This slope is therefore maintained and the concrete flows from each chute into a hopper and from this hopper it is poured into two-wheel barrows and wheeled to place. Tests of specimens of the concrete taken at different points in the system indicated that the strength was increased rather than diminished by flowing through a chute in this manner. The chuting on a uniform slope also requires a smooth mix of uniform consistency which will not separate. (See Fig. 2.)

MATERIALS, TESTS AND STRESSES.

Concrete Materials and Tests.—Although eastern Massachusetts is largely covered with glacial deposits of sand and gravel, it is by no means easy to find materials that make first-class concrete. Available sources of supply were sampled and tested before placing the contract for aggregates and finally a new bank located about 12 miles from the site was opened and the contract made for excavation, screening, and delivery.

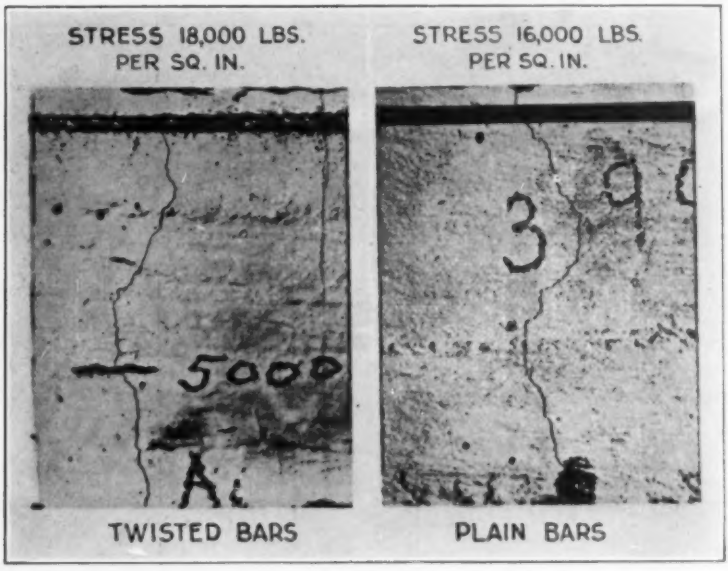

FIG. 3*a*.—CHARACTERISTIC CRACKS IN BEAMS REINFORCED WITH PLAIN AND TWISTED STEEL.

Immediately after deciding on the aggregate, compression tests of cylinders 8 in. in diameter by 16 in. long were made to determine the best proportions and allowable stresses. As a result 3 mixes were chosen, proportions 1 : 2 : 4, for the footings and foundation walls, 1 : $1\frac{1}{2}$: 3 for the superstructure, and 1 : 1 : 2 for a few important columns that required especially high stresses.

The cement used is tested regularly at the mill with check tests at the author's laboratory.

As sand tests are fully as important as cement tests, the

inspector goes to the bank at regular intervals for samples of sand which are sent to the author's laboratory for testing. The specifications required a ratio to standard sand of 70 per cent for 3 days and 85 per cent for 7 days. Although this is somewhat lower than the Joint Committee recommendations, it was found that the aggregate produced concrete of the required strength and furthermore the values could be used as absolute minimums on short time tests. As a matter of fact, the tests

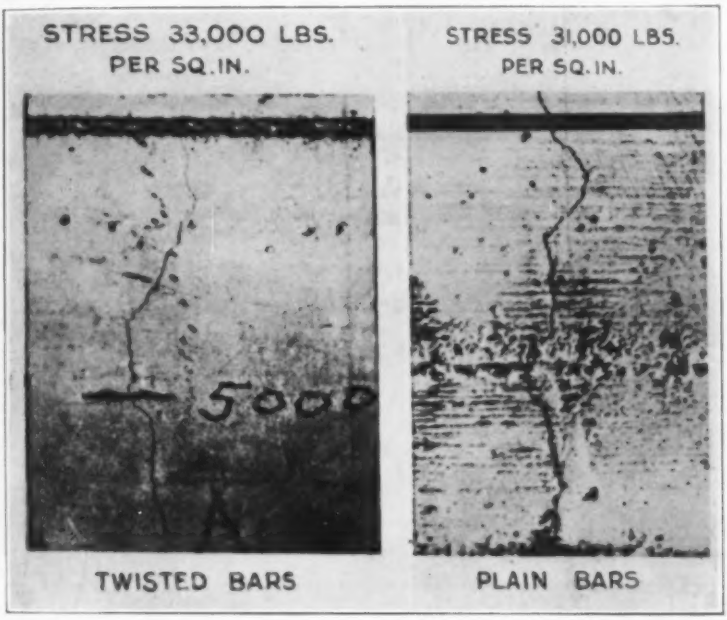

FIG. 3*b*.—CHARACTERISTIC CRACKS IN BEAMS REINFORCED WITH PLAIN AND TWISTED STEEL.

thus far have averaged 91 per cent for 3 days, 100 per cent for 7 days and 102 per cent for 28 days, an excellent record, especially for New England sands.

Field Tests.—Two field samples of concrete are taken regularly on each section of the job at least 4 times a week for test at 14 and 28 days, both samples from the same batch in order to arrive at the true relative value. Wherever possible the samples are taken directly from place, but for beams, columns, and girders,

this is impossible, and samples are taken from the barrow before placing. The concrete is shoveled into two 14-quart galvanized iron pails, carried to the molding yard, remixed to eliminate the segregation of materials due to carrying from the work, and poured into iron molds imbedded in moist sand. The maximum and minimum temperatures are recorded every day. For mass concrete where the aggregate runs up to 2 or $2\frac{1}{4}$ in., samples

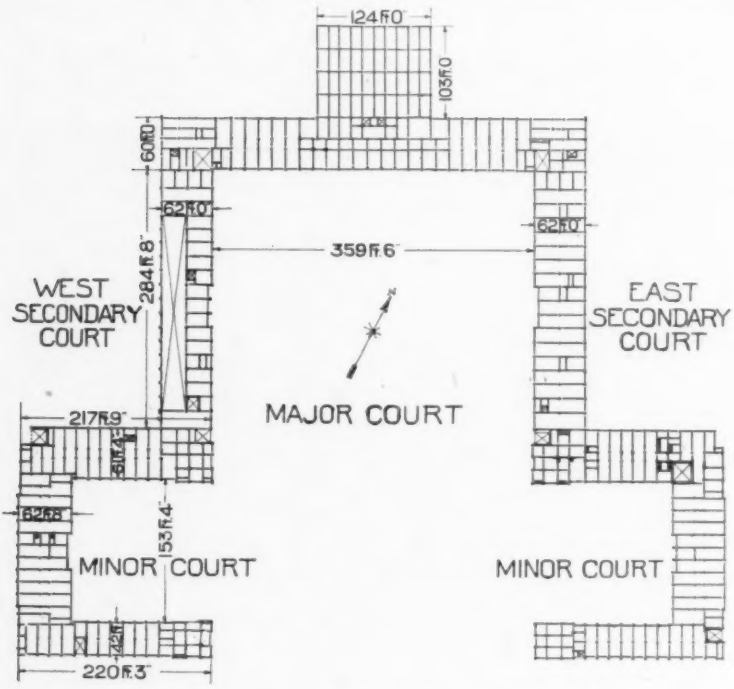

FIG. 4.—ARRANGEMENT OF BUILDINGS UNDER CONSTRUCTION AND FRAMING PLAN OF SECOND FLOOR.

are cast in 8 by 16 in. cylinders, but for the superstructure concrete, where the largest sized aggregate is not over $1\frac{1}{2}$ in., 6 by 6 by 12 in. prisms are used because the material for the larger size of specimen is too heavy to carry down from the upper stories. The bottom of each mold is formed by a $\frac{1}{2}$-in. iron plate. The concrete is tamped in the mold with a 6-in. ice chopper, taking about the same precautions as are employed on

the regular work. Each sample is given one tamp on each face and 4 tamps at 90 deg. in the center. The time of the operations of dumping into the molds and tamping never exceeds 5 minutes. Just previous to the initial set the top surfaces are carefully trowelled with a plasterer's trowel in order to obtain uniform bearing for the compression machine. After 48 hours the blocks

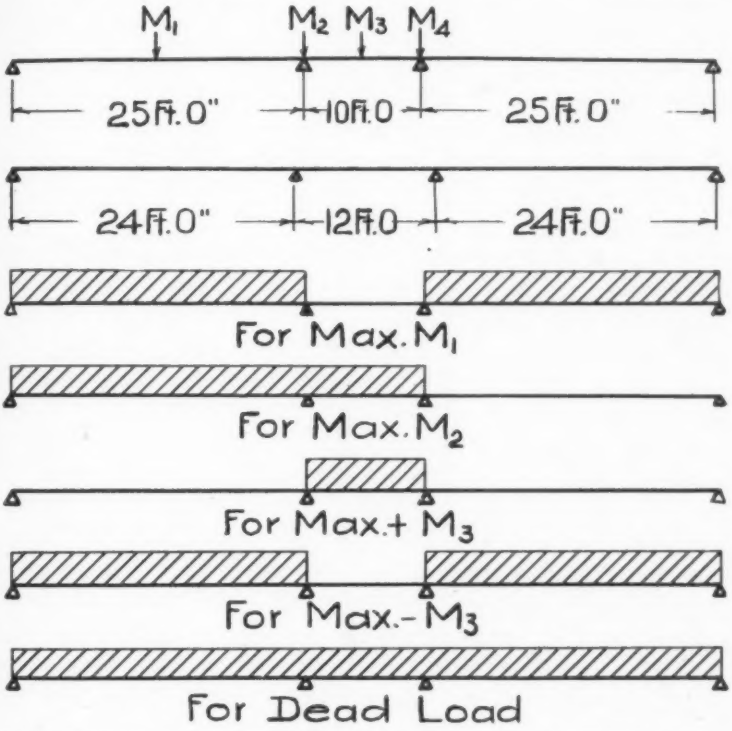

FIG. 5.—TYPICAL ARRANGEMENT OF BEAM SPANS AND DIFFERENT POSITIONS OF LIVE LOAD.

are taken out of the sand, removed from the molds, and reburied in the moist sand where they remain until the day before testing.

Results of Tests.—The average results of these concrete compression tests are as follows:

1 : 2 : 4	concrete,	14	days old,	1640 lbs.
1 : 2 : 4	"	28	" "	2110 "
1 : $1\frac{1}{2}$: 3	"	14	" "	2008 "
1 : $1\frac{1}{2}$: 3	"	28	" "	2520 "

Each value is an average of about 90 specimens and the per cent of variation from the mean runs much lower than is usual in construction work because of the care with which the samples were taken and molded.

The increase in strength from 14 to 28 days for the 1 : 2 : 4 concrete has averaged 23 per cent and for the 1 : 1½ : 3 concrete 25 per cent. The series of laboratory tests conducted last year by the Institute Committee on Aggregates showed an increase of 19 per cent in this period.

The practical value of this system of sampling and testing was shown at the beginning of the job. Some of the specimens of 1 : 2 : 4 concrete gave low test. The method of handling the concrete on the job was changed at once and subsequent specimens gave proper strengths. To be sure the concrete that had been previously placed was satisfactory, 3 or

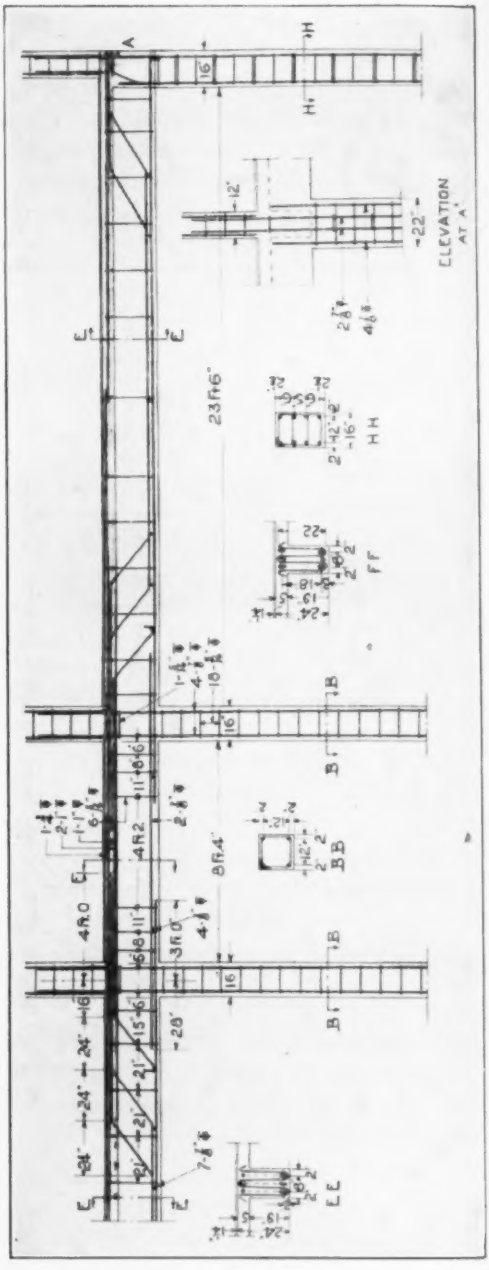

FIG. 6.—TYPICAL BEAM AND COLUMN, BUILDING ON MINOR COURT.

4 samples were cut out of a solid footing 3 or 4 weeks old. After being sawed to a standard 8 by 8 by 16 in. size, they were crushed and the strengths averaged 2200 lbs., a satisfactory figure for this portion of the structure.

Properties of Reinforcing Steel.—The cold twisted steel used for reinforcement was tested at the mill with the following results:

Yield point varied from 57,000 to 64,000 lbs. and the tensile strength from 70,000 to 81,000 lbs.

Elongation in 8 in. was 7½ per cent to 10 per cent and reduction in area was 33 per cent for the larger sizes of bars and 15 per cent elongation with 50 per cent reduction in area for sizes below ½ in.

The fracture was a half cup.

Bending tests for ½ in. and larger bars were 180 deg. around their own diameters and for bars below ½ in., 180 deg. flat.

Phosphorus ranged from .01 per cent to .02 per cent and sulphur from .03 per cent to .04 per cent.

Beam Tests.—The stresses were so chosen that no cracks would be visible under ordinary working loads and before coming to a decision on these stresses full size T-beams were made and tested.

As a result of the tests it was evident that the cracks in beams reinforced with twisted steel are less objectionable under a steel stress of 18,000 lbs. per sq. in. than cracks in beams reinforced with plain round bars under a stress of only 16,000 lbs. per sq. in. Moreover, so much of the tension was carried by the concrete that with a theoretical stress of 18,000 lbs. the actual measured stress is only 8000 lbs. In beams reinforced with twisted steel there were many more cracks than in beams reinforced with plain round bars, but these cracks were all small and well distributed, and therefore unobjectionable, whereas in the plain round bar beams the cracks were larger and more noticeable.

Fig. 3*a* and *b*, shows full size portions of cracks in two beams reinforced with plain round and with twisted bars. The crack on the right, Fig. 3*a*, was produced by a 16,000-lb. unit stress

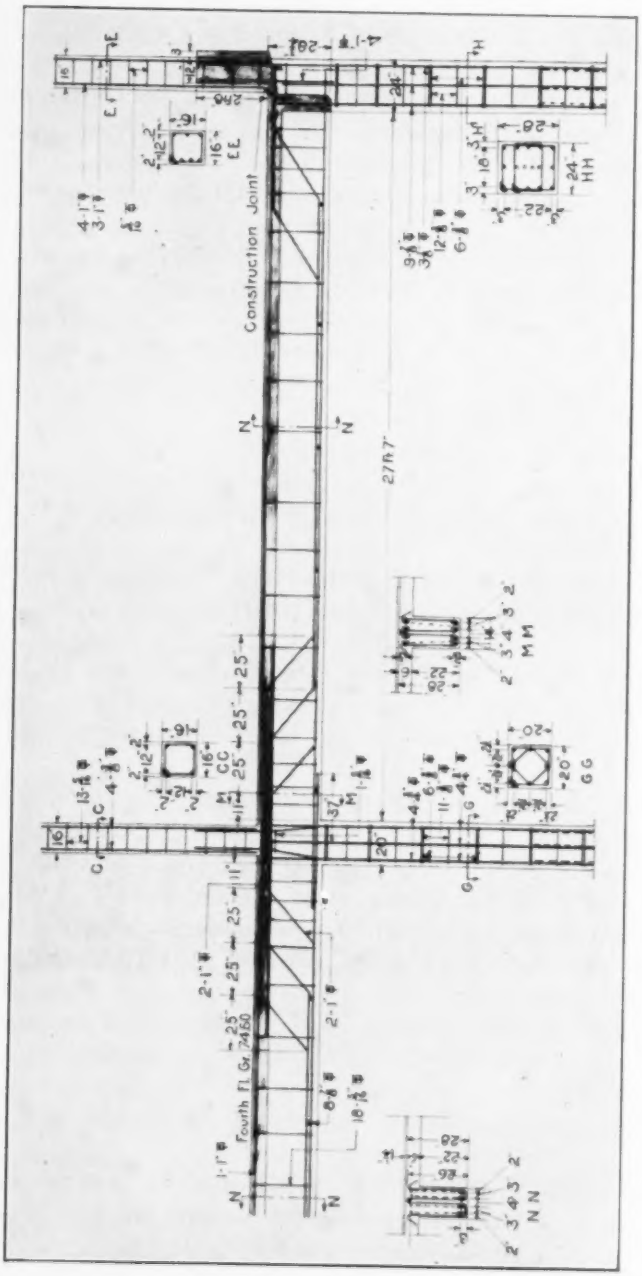

FIG. 7.—TYPICAL BEAM AND COLUMN, BUILDING ON MAJOR AND WEST SECONDARY COURTS.

in the round steel and that on the left by an 18,000-lb. stress in twisted steel.

Fig. 3*b* shows full size portions of cracks in two beams reinforced with plain round and with twisted bars but carrying about double the stress carried by the beams in Fig. 3*a*. The crack on the right was caused by a 31,000-lb. unit stress in the round steel and that on the left by a 33,000-lb. unit stress in the twisted steel. The stress of steel at failure for beams with twisted steel averaged 93,000 lbs. per sq. in. and for beams with plain round bars 60,000 lbs.

DESIGN.

Complete specifications, following substantially the recommendations of the Joint Committee on Concrete and Reinforced Concrete, were drawn up by the author. Only special features are considered in this paper.

UNIT STRESSES.

The following standard unit stresses are based on the compressive strength of 8 by 16 in. concrete cylinders, tested in the laboratory, using the percentages for different kinds of stress recommended by the Joint Committee on Concrete and Reinforced Concrete.

Item.	1 : 2 : 4 Concrete.	1 : 1½ : 3 Concrete.	1 : 1 : 2 Concrete.
Compressive unit strength of concrete in 8 by 16 in. cylinders	2000	2500	3300
Compressive fiber stress:			
At center of beam	650	810	....
At support	750	930	....
Compressive stress	450	560	750
Shearing unit stress	120	150	....
Bond stress	80	100	....
Ratio of elasticity	15	12	10
Steel tensile stress	18,000	18,000	18,000

TYPICAL BENDING MOMENTS.

In the layout of the beams (see Fig. 4) there is usually one short span between 2 long spans. This arrangement gives a series of class rooms on each side of a long corridor with most of the beams in the partition.

The line drawing, Fig. 5, shows the positions of live and dead load for maximum bending moments for this type of beam. The formulas from which the bending moments are derived are rather complicated and involve integral calculus. Solutions were worked out for the different conditions and constants obtained by use of which the general formulas were reduced to the simple ones given below.

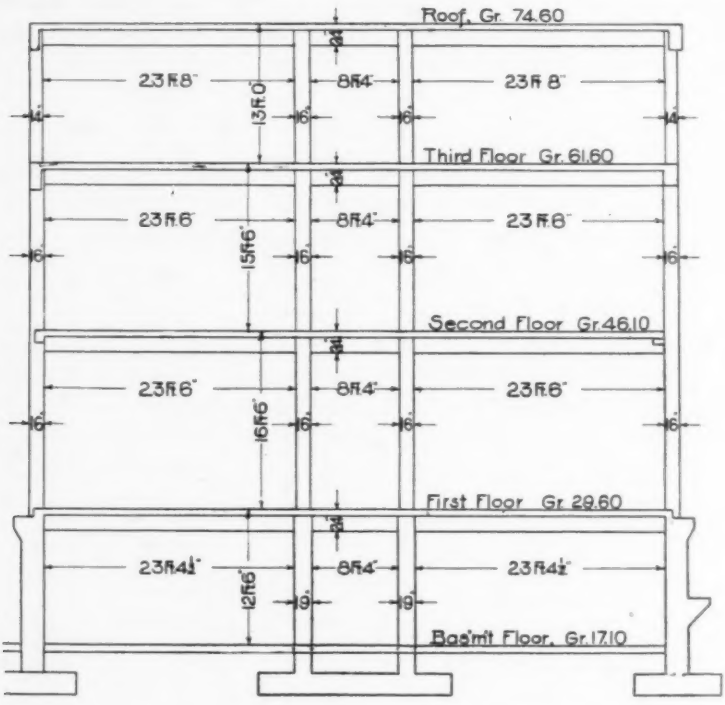

FIG. 8.—CROSS SECTION, BUILDING ON MINOR COURT.

Let

M_2 and M_4 = bending moments at center supports.

m_{x_1} and m_{x_3} = static bending moment at any point in the respective end span due to load in that span.

m_{x_2} = static bending moment at any point in the center span due to load in that span.

s = ratio of end span to center span.

$$2M_2(l+nl)+M_4nl=-6\left[\frac{1}{l}\int_0^{l} m_{x_1}\,x\,dx+\frac{1}{nl}\int_0^{nl} m_{x_2}(nl-x)\,dx\right]$$

$$M_2nl+2M_4(nl+l)=-6\left[\frac{1}{nl}\int^{nl} m_{x_2}\,x\,dx+\frac{1}{l}\int^{l} m_{x_3}(l-x)\,dx\right]$$

These formulas were solved and constants in terms of dead and live loads derived for different ratios of short span to long span and for different positions of the load.

BENDING MOMENT COEFFICIENTS.

Ratio of Short to Long Span.	Max. M_1		Max. M_2		M_3		
	Dead Load.	Live Load.	Dead Load.	Live Load.	Dead Load.	Live Load.	
						Positive.	Negative.
0.3	.084	.085	−.090	−.101	−.077	.009	−.086
0.4	.087	.089	−.083	−.096	−.063	.015	−.078
0.5	.087	.092	−.082	−.095	−.049	.022	−.072

For design, as a result of these studies, the following simple formulas were used for the three span continuous beam:

End spans: Center $M_1=\frac{1}{11}wl^2$

Support $M_2=-\frac{1}{11}wl^2$

Center spans: Supports $M_2=-\frac{1}{11}wl^2$

Middle $\begin{cases} M_3=\frac{1}{60}wl^2 \\ M_3=-\frac{1}{18}wl^2 \end{cases}$

In all above cases, l is the length of the end span.

RELATIVE ECONOMY OF DIFFERENT ARRANGEMENT OF BEAMS AND GIRDERS.

Before deciding on a flat slab layout with no intermediate beams, requiring 15 by 27 ft. slabs, studies were made of the

relative costs of this construction compared with intermediate beam and tile construction. The long span concrete slab construction proved most economical, however, when the total cost and estimates were made for the following cases:

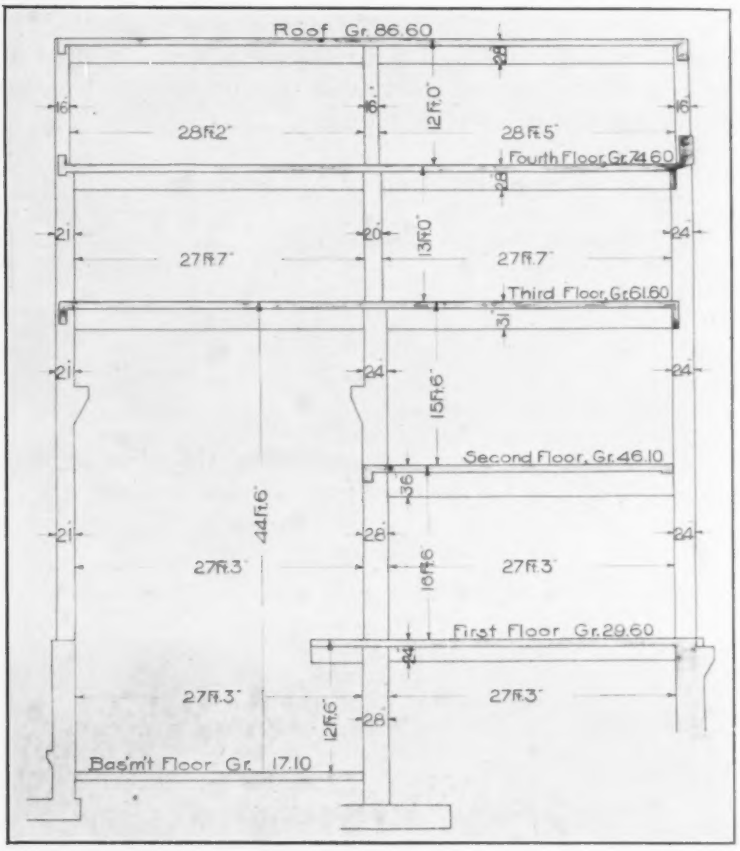

FIG. 9.—CROSS SECTION, BUILDING ON MAJOR AND WEST SECONDARY COURTS.

Case I. Panel with no intermediate beam; slab 15 ft. 6 in. solid concrete construction.

Case II. Panel with one intermediate beam; slab 7 ft. 9 in.

Case III. Panel with no intermediate beam; slab 15 ft. 6 in. with tile.

In Table I is summed up the relative costs for the 3 cases. It is evident that cost of forms is the important factor, completely upsetting the conclusions that would be drawn from the relative costs of the materials actually used in the slab.

Beam Reinforcement.—Beams are all continuous and negative moments were provided for in the design. The location of bends in steel was always selected so as to leave as much steel as was required for both tension and compression stresses. As a rule, about $\frac{3}{8}$ of the shear was considered to be carried by the bent

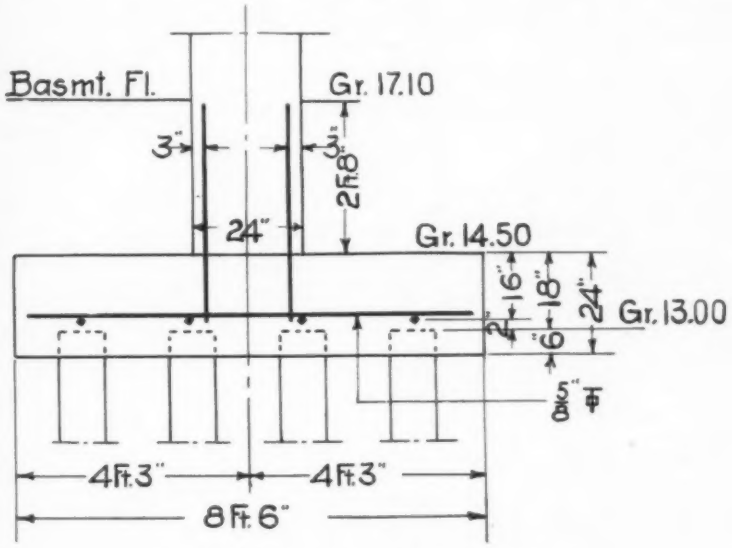

FIG. 10.—ELEVATION—INTERIOR FOOTING.

bars in accordance with recent German tests. Typical reinforced beams and columns are shown in Figs. 6 and 7. Typical cross sections of the buildings are shown in Figs. 8 and 9.

Unusually long beams of special design were used in the construction of the pavilions in Buildings 1, 2, 5, and 6, where it was desirable to have class rooms over 40 ft. sq. with no inside columns. Four beams running at right angles to each other, 2 in each direction, divided the floor into 4 nearly square panels. The beams were 18 by 36 in. and the reinforcement consisted of six 1-in. and six $\frac{7}{8}$-in. bars at the center and five 1-in. and two

TABLE I.—RELATIVE COST OF DIFFERENT SLAB DESIGNS.

Item.	Unit Cost.	Concrete Slab. Span 15 ft. 6 in.		Concrete Slab. Span 7 ft. 9 in. 1 Intermediate Beam.		Hollow Tile. Span 15 ft. 6 in.		Page Reference "Concrete Costs."
		Amount.	Cost.	Amount.	Cost.	Amount.	Cost.	
Concrete per panel:								
Slab.....cu. yd.	....	10.7	....	6.1	....	6.6	....	....
Beam.....cu. yd.	....	2.8	....	2.6	....	2.2	....	....
Ginder.....cu. yd.	....	....	....	0.65	....	....	....	....
Total.....	$5.82	13.5	$78.50	9.35	$55.00	8.8	$51.22	....
Steel per panel:								
Slab.....lb.	....	1080	....	435	....	607	....	....
Beam.....lb.	....	944	....	1620	....	944	....	....
Girder.....lb.	....	....	....	690	....	....	....	....
Total.....	0.027	2024	55.60	2745	75.00	1551	41.88	....
Tile per panel:								
Slab.....piece	0.200	....	....	....	....	286	57.20	....
Total cost of concrete.....	....	....	**$134.10**		**$130.00**		**$150.30**	....
Form Lumber:								
Slab.....1¼ in. stock ft. B.M.	....	1772	....	1435	....	1772	....	617
Beam....1½ in. stock ft. B.M.	....	496	....	680	....	429	....	619
Girder...1½ in. stock ft. B.M.	....	....	....	267	....	....	....	619
Total for 4 floors.....	....	2268	....	2382	....	2201	....	....
Total per floor per panel..M.	$30.00	567	$17.00	596	$17.86	550	$16.50	....
Form Labor:								
Slab.....		....	....	....	....	....	....	644
Make.....		$4.39	....	$3.78	....	$4.39	....	....
Place and remove first floor..		12.88	....	11.48	....	12.88	....	....
" " " second " ..		9.88	....	8.57	....	9.88	....	....
" " " third " ..		11.02	....	9.31	....	11.02	....	....
" " " fourth " ..		11.02	....	9.31	....	11.02	....	....
Total cost 4 floors.....		$49.19	....	$42.45	....	$49.19	....	....
Cost per panel per floor.....		....	$12.30	....	$10.61	....	$12.30	....
								639
Beams:								
Make.....		$3.00	....	$2.10	....	$2.70	....	....
Place and remove first floor..		5.79	....	4.40	....	5.10	....	....
" " " second " ..		5.20	....	3.90	....	4.60	....	....
" " " third " ..		6.92	....	5.10	....	6.80	....	....
" " " fourth " ..		6.92	....	5.10	....	6.80	....	....
Total cost per beam 4 floors..		$27.83	....	$20.60	....	$26.00	....	....
Cost per panel per floor.....		....	$6.95	....	$10.30	....	$6.40	....
Girders:								641
Make.....		....	....	$2.30	....	....	....	....
Place and remove first floor..		....	....	4.80	....	....	....	....
" " " second " ..		....	....	4.10	....	....	....	....
" " " third " ..		....	....	6.20	....	....	....	....
" " " fourth " ..		....	....	6.20	....	....	....	....
Total cost 4 floors.....		....	....	$23.60	....	....	....	....
Cost per panel per floor.....		....	....	....	$9.50*	....	....	....
Total cost forms.....	....	....	**$36.25**	....	**$48.27**	....	**$35.20**	....
Total cost.....	....	....	**$170.35**	....	**$178.27**	....	**$185.50**	....

Carpenter labor, 50c. per hour. Ordinary labor, 25c. per hour.

* Multiply cost per panel per floor by 1.6 instead of 2.0 to allow for spandrel beam.

Note that cost of forms and not cost of materials used in beams and slab determines relative cost of the three alternate designs.

Forms assumed to be used four times and remade once for a change in size of columns. Costs are from "Concrete Costs" by Taylor and Thompson according to page numbers given. Size of beam below slab, 12 in. by 19 in. Size of girder below slab, 14 in. by 19 in.

$\frac{7}{8}$-in. bars at the ends. All bars were square twisted steel. The pavilion roof beams are also unusually large because of the architectural treatment. In Building 17 there is a 48-ft. beam 24 by 60 in., reinforced with eighteen 1-in. bars at the center and twelve 1-in. bars at the end. In this case it was necessary to strengthen the support by haunches.

Interior Columns.—All interior columns are square and reinforced with vertical steel only. Bands were placed 12 in. on centers to keep the steel in position during construction. The eccentricity of interior columns was neglected because the short beams were designed with sufficient steel to prevent bending in the column.

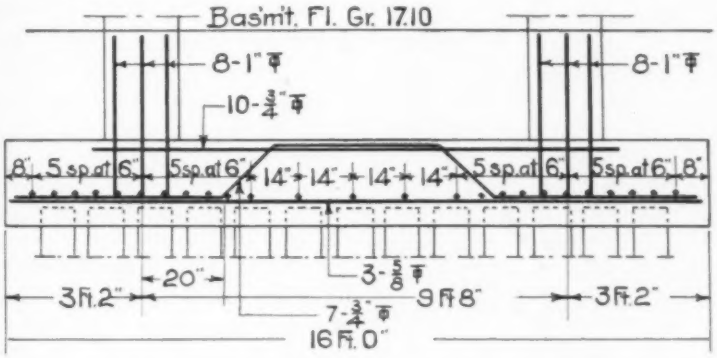

FIG. 11.—SECTION—TYPICAL BAY—EAST WALL FOOTING

Wall Columns.—In the design of wall columns account was taken of the bending moment produced by the rigidity of the connection between beam and column. The eccentricity in this case was computed carefully and enough steel provided to take care of it. To provide for the bending moment in the top story, however, it would have been necessary to use larger columns than the architectural treatment would permit. Therefore, to prevent a bending moment in the columns, it was decided to make a joint between the columns and roof beams and not to carry the outside reinforcement through the beam. Columns were then designed for the superimposed load placed on the column with an eccentricity of 0.4 the width of the column.

Slabs.—The floor slabs, which are continuous, were designed for a bending moment $\frac{wl^2}{12}$ at the support and the same moment in the center of the span. To prevent cracks in the building as far as possible, short bars were placed on the top over each girder whenever the main slab reinforcement ran parallel to the girder, and, in fact, wherever there was danger of negative stresses.

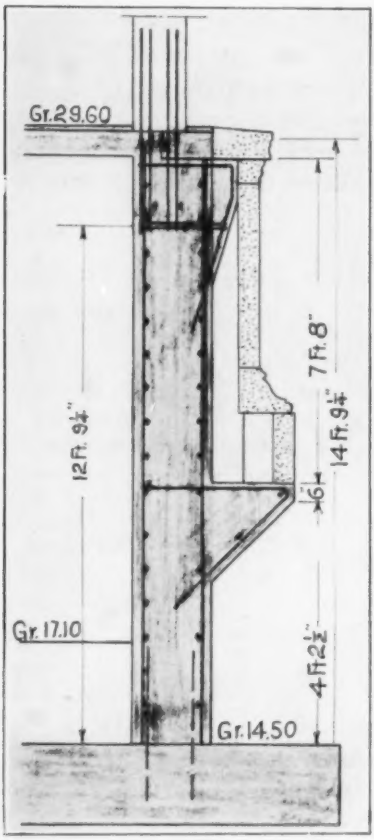

FIG. 12.—CROSS SECTION OF BASEMENT WALL.

Walls.—Retaining walls were cast monolithic with the wall columns except on the outside wall. In the minor and major courts separate retaining walls were built about 3 ft. 9 in. from the building line. Typical dimensions and reinforcement are shown in Fig. 12.

Footings.—The design of footings varies in different parts of the structure. In certain buildings the interior footings are continuous; in others individual footings are used, and in still others two interior columns are placed on one footing. All outside footings are continuous. Footings rest on wood piles of spruce or oak, depending upon the character of the ground, as determined by thorough tests. Figs. 10 and 12 give a general idea of the design and reinforcement of the footings.

Floor Surfaces.—Special studies of floors were made and several experimental slabs were built in order to find the method of placing that would result in minimum dusting and a satisfactory bond with the slab. The scum which always rises on

the surface of base concrete was removed by brushing the slab with a wire brush just as it begins to harden. The tests showed that dusting is best prevented by using coarse sand containing scarcely any dust and mixed with broken stone ranging from $\frac{1}{4}$ in. to $\frac{1}{2}$ in. in size and mixed in a stiff mortar in the proportions $1 : \frac{3}{4} : 1\frac{1}{4}$. As a result of laboratory tests, stiff neat cement brushed into the set concrete was selected as the best bonding material between the base concrete and granolithic.

The architect is William W. Bosworth; the engineers and builders, Stone & Webster Engineering Corporation; and the consulting engineers are Charles T. Main, on the footings; Sanford E. Thompson, on the concrete superstructure; J. R. Worcester & Co., on the structural steel; and Hollis French & Allen Hubbard, on the heating and ventilating system.

DISCUSSION.

Mr. Perrot. MR. E. G. PERROT.—How thick is the finish coating?

Mr. Thompson. MR. SANFORD E. THOMPSON.—An inch and a quarter.

Mr. Whited. MR. WILLIS WHITED.—There is one important innovation in the design. A richer concrete than usual was used. That is considerably cheaper than the leaner concrete; but I was always afraid to use it because of the shrinkage in setting.

Mr. Thompson. MR. THOMPSON.—It has worked out very satisfactorily. We have some very fine work. Of course you get a greasier mix with the rich proportion.

Mr. Whited. MR. WHITED.—It makes a better reinforcement than a leaner concrete, does it not?

Mr. Thompson. MR. THOMPSON.—Yes; and although the whole structure was built practically as a monolith we have had no shrinkage cracks except a very few temperature cracks which would have appeared in any case.

Mr. Deberard. MR. DEBERARD.—Is any provision made to prevent floor dusting?

Mr. Thompson. MR. THOMPSON.—That was one of the principal reasons for these tests and the plan is to use a very stiff mixture, in which the fine material will not come to the surface, and to practically eliminate fine material; that is, to use a sand scarcely any of which will pass a 30-mesh sieve. We use a Plum Island sand, as it is called in Boston. There may be a very fine dust when the concrete is made like that, but it will be so very fine that it will not be objectionable and will wear off very quickly, getting down into the base of the concrete.

Mr. Perrot. MR. PERROT.—In designing columns where the bays are 10 to 12 ft. wide and from 24 to 25 ft. long, was eccentricity taken into account?

Mr. Thompson. MR. THOMPSON.—It was. I am glad you spoke of that point. The eccentricity was computed carefully and enough steel was provided to take care of it. It was impossible, however, from an architectural standpoint, to use the large sized column required for this in the top story, and, therefore, what was substantially

a horizontal joint was made on the top columns, so that there would be no bending moment. Mr. Thompson.

MR. T. L. CONDRON.—I notice that the columns were not spirally wound. Mr. Condron.

MR. THOMPSON.—They were not. Mr. Thompson.

MR. CONDRON.—It was recently discovered in case of fire that a square column tends to become round. Mr. Condron.

MR. THOMPSON.—In buildings like those of the "Institute of Technology" of course there is comparatively little inflammable material, and I am of the opinion that the square column, except under such extreme conditions as were confronted in the Edison Fire, is good design. Mr. Thompson.

MR. CONDRON.—In the Edison Fire, spalling of square columns, which had very considerably mitered corners, occurred almost without exception, wherever there was fire. In places where the heat was not sufficient to destroy the insulation on electric wires strung through the ceiling, square columns were spalled almost as badly as those in the rest of the building where the heat was exceedingly great. In fact, there were large areas in which it was found necessary to chip off the corners of the columns where no repairs were necessary to other members. The fireproofing on the under side of the beams had not been effected seriously, but invariably the columns were cracked and the corners loosened. The loosening of the corners was not effective in more than 50 per cent of the cases, such loosening did not extend to the vertical reinforcement, which, by the way, was placed 3 in. or more from the faces of the columns. Mr. Condron.

DESIGN OF WALL COLUMNS AND END BEAMS.

By Edward Smulski.*

The question of proper design of wall columns and end beams is very important, but it does not receive in practical design the attention that it deserves. As a consequence, in many structures, wall columns have a much smaller factor of safety than the rest of the structure. The purpose of this paper is to supply information which could be used by the designer, but also to create a discussion which may throw more light on the subject.

In ordinary practice, the beams and columns are either built as monoliths, or they are connected by steel running horizontally from beam into the column and vertically from column into the beam, so that the bending moment from one member is transferred to the other member. Therefore the columns and end beams must be designed according to the following suggestions. As a rule, this is not very difficult except in the top columns where the bending moments are very large in comparison with the superimposed load carried by the column.

Recommendations for Design.

Bending Moments.—The bending moments developed in the end beam and transferred to the column will vary with different conditions. Their size is dependent upon the ratio of moments of inertia of beam and column, and also upon the ratio of length of beam to height of column. The table following gives the moment coefficients for different ratios of moments of inertia and different ratios of length to height. An appropriate value may be selected by the designer for any condition.

For ordinary conditions, the negative bending moment in the beam at the wall column will be in the vicinity of $\frac{wl^2}{18}$. When a beam frames into a large column, the ratio of moment of inertia of beam to that of the column will be smaller than unity and the

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value of the negative bending moment approaches more and more the limiting value, $\frac{wl^2}{12}$. On the other hand, if a deep beam is supported by a comparatively slender column, the negative bending moment developed in the beam is comparatively small, as the column does not offer any considerable restraint for the beam.

End Beams.—The beams entering wall columns and built monolithic with them can be considered as partially continuous. The negative bending moment at the support must be provided for, the coefficient for which may be selected from the table. If the beam is properly designed at the wall support and the column designed according to the following suggestions, the positive bending moment in the center and the negative bending moment at the interior support of the end beam may be taken as given in the table; otherwise, the bending moment, $\frac{wl^2}{10}$, shall be used.

Column.—The column must be designed for direct stress and the bending moment according to the formulas for direct stress and flexure. (See Taylor and Thompson's "Concrete Plain and Reinforced," pp. 564–574.)

Direct stress is the stress caused by the total load supported by the column.

The bending moment is the total of all bending moments to which the column is subjected. This consists of the bending moment produced by the end beam entering the column and the bending moments, if any, caused by eccentric application of other loads like brickwork in the spandrels or parapet.

In the top story, the total bending moment from the end beam must be considered as taken by the column, while in the lower stories, half of the bending moment may be considered as taken by the column below in negative bending moment and half by the column above in positive bending moment.

Sometimes the spandrel beam is placed eccentrically. If it is nearer the outside face of the column, the bending moment is of opposite character than the bending moment caused by the beam, and should be deducted, while if nearer the inner face, the bending moment should be added.

Allowable Unit Stresses.—The allowable compressive unit stresses in the beam at the support may be assumed 15 per cent

larger than in the center of the span because the bending moment reduces rapidly and the maximum stress acts only for very short length.

The maximum allowable combined stresses in the column due to flexure and direct stress, found from formulas previously referred to, may be assumed much larger than the allowable stresses for axial loading or direct compression only, because the stresses are in the nature of fiber stresses, that is, they are not uniformly distributed over the whole cross section, but are maximum at the edge and decrease according to a straight line. The bending moment in the column reduces rapidly, and therefore similarly as in the beam, the maximum stress acts only in a short length and on a small part of the cross section. The same unit stress, therefore, may be allowed as in a beam at the support, with the understanding, however, that the uniform unit stress caused by the superimposed load alone does not exceed the allowable unit stress for axial loading. For 1:2:4 concrete of an ultimate strength of 2000 lb., 450 lb. per sq. in. is allowable as direct stress; 650 lb. per sq. in. is allowable for fiber stress; and 650+15 per cent is allowable for the beam at the support and also the maximum combined stress in the column.

Wall Column Supporting Flat Slabs.—In a flat slab construction, it is not necessary to provide in the wall column for the total bending moment at the column-head because part of the bending moment developed on one side of the column-head is neutralized by the bending moment developed on the other side of the column-head. We may, therefore, assume that the bending moment to be resisted by the column should be taken as $\frac{3}{5}$ of the bending moment developed around the wall column-head.

The negative bending moment at the wall column-head to be resisted by the steel in the slab shall be taken as $\frac{2}{3}$ of the negative bending moment figured in the interior panels for $\frac{1}{2}$ of the column-head.

Exceptions.—It must be noticed that the wall columns are not always subject to bending moment. For instance, beams may run lengthwise with the building so that all the load would come from the wall beams placed on both sides of the column.

As a general proposition, it may be stated that any column receives bending moment when a beam runs into it on one side

only, that is, when the bending moment produced by the beam is not balanced by another beam coming from the opposite direction.

EXPLANATION OF THE RECOMMENDATIONS.

The necessity for the above recommendations is evident from the following consideration of the subject.

There exists a law in mechanics which unfortunately is not appreciated as generally as is desirable, that stresses in structural members go hand in hand with deflections and deformations. Deflection cannot exist without stresses in the material any more than stresses can exist without consequent deformation in the material. If we apply this law to wall column and wall beam, we will see that negative bending moments must be developed at the end of a beam connected with wall column and that the bending moments in the beam must be transferred to the column.

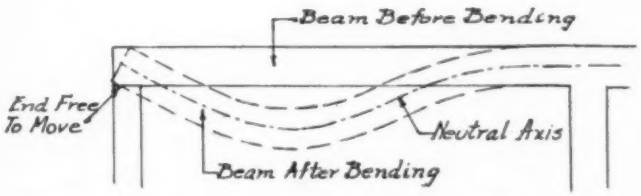


FIG. 1.—DEFLECTION OF BEAM SIMPLY SUPPORTED AT WALL COLUMN.

An end beam supported simply at the wall end and continuous on the other end under load will deflect, as shown exaggerated in Fig. 1. It is evident from the figure that after deflection takes place, the end of the beam does not bear uniformly on the column and therefore the load is transferred eccentrically to the column. But as there is no connection between the beam and the column, no bending moment can be developed at the end support, and therefore no bending moment is transferred from the beam to the support.

In Fig. 2 is shown a beam connected with the column. The end is restrained from motion, therefore the beam under load would assume a different shape than in the first case. In order to allow the center to sag, the deflection curve must assume the

shape of a reversed curve, as shown in Fig. 2. The points of inflection of this curve will vary with the restraint offered by the column. The deflection is accompanied by bending moments; therefore, we have in the center of the beam a positive bending moment, and at the supports, negative bending moments. At Point "a," this bending moment is maximum and decreases to 0 at the point of inflection where it changes its sign.

The deflection at "a" in the beam cannot disappear, but must cause similar bending in the column. This deflection causes bending stresses in the column. The shape of the column after deflection will depend upon whether the column is in the top

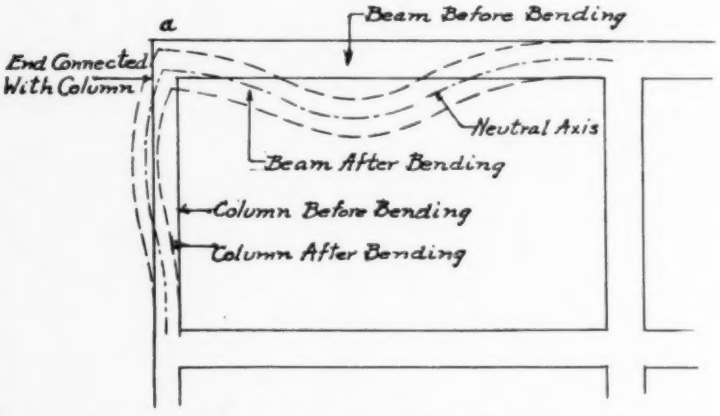


FIG. 2.—DEFLECTION OF BEAM MONOLITHIC WITH WALL COLUMN.

story or in the intermediate stories. If the column is in the top story, the total negative bending moment from the beam is taken directly by the column, while in lower stories, the bending moment from the beam is partly taken by the column below as negative bending moment, and partly taken by the column above as positive bending moment. Therefore, for the same conditions of loading and the same relative sizes of beams and columns, the bending moment in the top column is twice as large as in the intermediate columns.

Sometimes it is argued that by putting somewhat more steel into the beam at the center, the deflection of the column can be prevented. This is not the case. By the increase of steel, the

moment of inertia of the beam is increased only a little. As the ratio of moments of inertia will be changed inappreciably, the negative bending moments at the wall support and in the column will change very little.

As evident from Fig. 2, the column will bend as long as there is a connection between the beam and the column. When not designed right, the bending moment will cause cracks on the outside of the column and, what is more important, will develop excessive compressive stresses. These are specially dangerous when the concrete is green. Numerous erection failures can be traced to this cause.

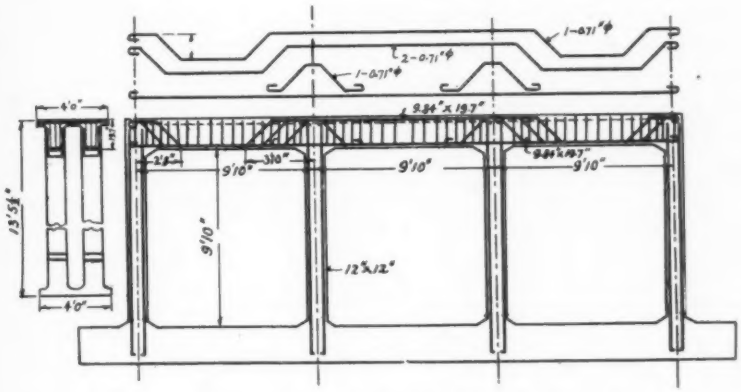


FIG. 3.—DIMENSIONS AND REINFORCEMENT OF TEST BEAM.

TESTS.

Unfortunately very few tests of beams connected with the columns are available. The best test was made in Germany by Professors H. Scheit and E. Probst.*

Test by Scheit and Probst.—The test consisted of a beam of 3 spans supported on columns shown in Fig. 3. As noted from the figure, the beams and columns were designed in the same way as is customary in ordinary construction, that is, no special precautions were taken to make the connections between column and beam rigid.

* Untersuchungen an durchlaufenden Eisenbetonkonstruktionen.

Dimensions: Beam span 9 ft. 10 in.; depth, 19.7 in.; width of stem, 9.84 in. Column height, 9 ft. 10 in.; section 12 by 12 in.

Concrete used: 1 part cement by volume, 5 parts aggregates, consisting of broken stone and sand, 9 per cent water by weight.

Compressive strength of cubes, 3700 lb. per sq. in., transverse strength, 425 lb. per sq. in.; tensile strength, 310 lb. per sq. in.

Steel: Elastic limit, 45,000 lb. per sq. in.; tensile strength, 65,000 lb. per sq. in.

Age of test, 45 days.

Two substantially identical specimens were tested. During testing, the end spans only were loaded. The load was applied at 8 points, so that it may be considered as uniformly distributed. The load was applied gradually with an increment of about 550 lb. per lin. ft.

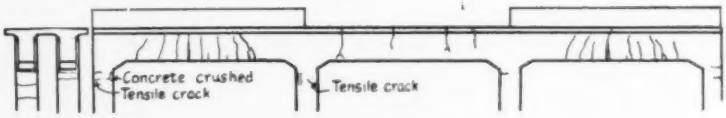


FIG. 4.—CRACKS IN BEAMS AND COLUMNS UNDER LOAD.

The cracks appeared in the following order:

First crack at 4550 lb. per lin. ft. in the center of the end span.

First crack in center of middle span at the top, due to the negative bending moment, at 10,900 lb. per lin. ft.

First negative cracks at the column, 14,000 lb. per lin. ft.

These last cracks extended almost to the center of the column. Cracks in the interior column at the top developed at 16,800 lb. per lin. ft. At 17,600 lb. per lin. ft., the crack in the center of the end span increased and at the same time signs of compressive failure at the inside edge of the end column appeared. The two causes resulted in final failure.

Fig. 4 shows the cracks at maximum load.

If we figure the stresses in this test by means of least work, we find that the ratio of moments of inertia of beam and column

is 3:7, and the corresponding negative bending moment at the column is about $-\frac{1}{32} wl^2$, which agrees with the value given in the table for the respective ratio of moments of inertia.

The maximum positive bending moment was $\frac{wl^2}{12.1}$.

For maximum load of 17,600 lb. per lin. ft., the stresses in steel based on above bending moment are as follows:

In center of the end beam, $f_s = 46{,}200$ lb. per sq. in.
In the column, $f_s = 23{,}200$ " " " "

These figures correspond pretty closely with actual stresses.

Using the same formulas, we find that at the first cracks the maximum tension in concrete coincided very closely with the values obtained for transverse strength of concrete.

The foregoing test shows conclusively that the bending moments from the loaded beam are transferred to the unloaded beams and the columns as long as there is any connection whatever between the column and the beam. The bending moment developed in the column, of course, as it depends entirely upon the relation of the moment of inertia in the column and in the beam.

It is of interest to notice the comparison between this test and the test of a continuous beam of exactly the same design as previously described but supported on knife edges. The first crack in the latter case appeared at an average of 2560 lb. per lin. ft. in the center of the end span (as compared with 4550 lb. in previous case). In the middle span, the first crack appeared at 5600 lb. per lin. ft. Final failure occurred at 12,600 lb. per lin. ft. against 17,600 lb. per lin. ft. in the case where beam was connected with columns.

Slater Test.—Another proof that bending moments are transferred to the column is offered by Mr. Slater's test described in his paper on "Test of Flat Slab Floor," Shredded Wheat Factory, Niagara Falls, N. Y., read before this Institute. Measurements were taken in the column showing decided bending in the wall column. For particulars, refer to the paper.

MOMENT COEFFICIENTS α FOR END BEAMS CONNECTED WITH WALL COLUMNS.

To be used in Formula $M = \alpha \frac{wl^2}{d}$.

I_1 = moment of inertia of beam.
I = moment of inertia of column.
$\frac{I_1}{I}$ = ratio of moments of inertia.
l = length of beam.
h = height of column.
$\frac{l}{h}$ = ratio of length to height.
w = uniform load per lin. ft. of beam.

$\frac{I_1}{I}$	Ratio of Length to Height $\frac{l}{h}$ = 1		2		3		4		5	
	Bending Moment at Wall Column.	Bending Moment at Center and at Interior Column.	Bending Moment at Wall Column.	Bending Moment at Center and at Interior Column.	Bending Moment at Wall Column.	Bending Moment at Center and at Interior Column.	Bending Moment at Wall Column.	Bending Moment at Center and at Interior Column.	Bending Moment at Wall Column.	Bending Moment at Center and at Interior Column.
0.5	$\frac{1}{15}$	$\frac{1}{12}$	$\frac{1}{14}$	$\frac{1}{12}$	$\frac{1}{13}$	$\frac{1}{12}$	$\frac{1}{13}$	$\frac{1}{12}$	$\frac{1}{12}$	$\frac{1}{12}$
1	$\frac{1}{18}$	$\frac{1}{12}$	$\frac{1}{15}$	$\frac{1}{12}$	$\frac{1}{14}$	$\frac{1}{12}$	$\frac{1}{14}$	$\frac{1}{12}$	$\frac{1}{13}$	$\frac{1}{12}$
2	$\frac{1}{24}$	$\frac{1}{11}$	$\frac{1}{18}$	$\frac{1}{12}$	$\frac{1}{15}$	$\frac{1}{12}$	$\frac{1}{15}$	$\frac{1}{12}$	$\frac{1}{14}$	$\frac{1}{12}$
4	$\frac{1}{36}$	$\frac{1}{10}$	$\frac{1}{23}$	$\frac{1}{11}$	$\frac{1}{20}$	$\frac{1}{11}$	$\frac{1}{18}$	$\frac{1}{12}$	$\frac{1}{17}$	$\frac{1}{12}$
10	$\frac{1}{72}$	$\frac{1}{10}$	$\frac{1}{44}$	$\frac{1}{10}$	$\frac{1}{32}$	$\frac{1}{11}$	$\frac{1}{27}$	$\frac{1}{11}$	$\frac{1}{25}$	$\frac{1}{11}$

NOTE.—Bending moment at wall column and interior column, negative.
Bending moment in center of beam, positive.

REPORT OF COMMITTEE ON REINFORCED CONCRETE AND BUILDING LAWS.

Tests of Reinforced Concrete Columns.

I. Preliminary.

Your Committee has made on behalf of the American Concrete Institute the herein reported tests of reinforced concrete columns of large size.

Acknowledgments.—In the execution of these tests the Institute has been favored with the co-operation of the United States Bureau of Standards in the use of the ten million pound compression testing machine at Pittsburgh, Pa., and of the Engineering Experiment Station of the University of Illinois in furnishing observers in conducting the tests. The tests were made possible through the active co-operation of those who donated the funds and materials for making and molding the columns.

The test columns were fabricated in New York by the Turner Construction Company. The making of the columns was inspected by A. S. Merrill, Assistant to the Chief Engineer of the Turner Construction Company, and D. A. Abrams, of the Engineering Experiment Station of the University of Illinois. Mr. Merrill rendered valuable service as general observer and recorder and in preparing the data for publication.

The tests of the columns and cylinders were made in the Bureau of Standards Laboratory at Pittsburgh. Especial acknowledgment is due S. W. Stratton, Director of the Bureau of Standards, P. H. Bates, in charge of the Pittsburgh laboratory and J. H. Griffith, Engineer-Physicist.

Acknowledgment is also due to F. E. Foss of Cooper Union, New York, for laboratory facilities in making auxiliary tests.

A. N. Talbot represented your committee; W. A. Slater and H. F. Gonnerman of the Engineering Experiment Station of the University of Illinois supervised the preparation of the columns for the tests and were in direct charge of the testing operations.

The extensive data taken on these tests have been reduced

in the office of the Engineering Experiment Station of the University of Illinois, and in the office of the Turner Construction Company.

The cement was furnished by the Atlas Portland Cement Company; it was sampled by Robert W. Hunt and Company. Two of the samples were tested by that company and a third by the Allentown Testing Laboratory.

The sand and gravel were furnished by the Phoenix Towing and Transportation Company and came from Hempstead Harbor, Long Island. Strength tests on 1–3 briquettes made up with the sand and Vulcanite Portland cement were made by Sanford E. Thompson, who also reported on the concrete making qualities of the aggregate.

The spiral reinforcement was furnished by the American Steel and Wire Company and the longitudinal reinforcement by the Carnegie Steel Company.

II. SCOPE OF TESTS.

Twenty-four columns were tested, having a variety of reinforcement, both longitudinal and lateral. In the tests, measurements were made to find the deformation in various parts of the column, including the stretch in the spiral reinforcement. This set of tests is a part of a program laid out by the committee for the purpose of determining the influence of both spiral reinforcement and longitudinal reinforcement upon the structural properties of columns. The schedule of tests as given in the JOURNAL, Vol. I, No. 2, December, 1913, is here reproduced:

HOOPED COLUMNS ("Hooped" Applies to Columns With Spiral).

Group I. *Vertical steel the variable.*

Columns 1, 1a and 1b.	1 per cent spiral.	No vertical steel.
Columns 2, 2a and 2b.	1 per cent spiral.	1 per cent vertical steel.
Columns 3, 3a and 3b.	1 per cent spiral.	2 per cent vertical steel.
Columns 4, 4a and 4b.	1 per cent spiral.	6 per cent vertical steel.

Group II. *Effect of increasing spiral.*

Columns 5 and 5a.	2 per cent spiral.	1 per cent vertical steel.

Group III. *Effect of protective shell.*

Columns 6 and 6a.	1 per cent spiral.	1 per cent vertical steel.

Group IV. *Plain concrete.*

Columns 7 and 7a.	No spiral.	No vertical steel.

VERTICALLY REINFORCED COLUMNS.

Group V.

Columns 8 and 8a. 1 per cent vertical steel. ¼-in. diameter ties, 12 in. apart.

Columns 9 and 9a. 2 per cent vertical steel. ¼-in. diameter ties, 12 in. apart.

Columns 10 and 10a. 4 per cent vertical steel. Double ¼-in. diameter ties, 12 in. apart.

The range of variables in the test columns is given in Table I. Figs. 1 and 2 show the form and dimensions of the columns and the amount and position of the reinforcement. The shaft of the column for a length of 9 ft. 4 in. was generally about 21 in. in diameter. The two columns used to show the action of an outer protective coating were 24 in. in diameter. In the columns with spiral reinforcement, the four spacers had an area equal to about

TABLE I.—RANGE OF VARIABLES IN TEST COLUMNS.

Spiral Reinforcement, Nominal per cent.	Longitudinal Reinforcement, per cent.				
	0	1	2	4	6
0	7, 7a	8, 8a	9, 9a	10, 10a	
1	1, 1a, 1b	2, 2a, 2b 6, 6a	3, 3a, 3b		4, 4a, 4b
2		5, 5a			

0.2 per cent of the column section. This metal has not been considered in computing the percentages of longitudinal reinforcement. Ties ¼-in. in diameter were placed 12 in. apart in the columns without spiral reinforcement. They held the longitudinal reinforcement in position during the fabrication of the column.

The concrete was a $1 : 1\frac{1}{2} : 3$ mixture. With each column two cylinders 8 in. in diameter and 16 in. long were made for testing with the columns. Additional cylinders of the same size were made to determine the increase in the strength of the concrete with age.

III. MATERIALS.

Cement.—The cement was taken from shipments received by the Turner Construction Company for use in the construction of the warehouse at which the test columns were made.

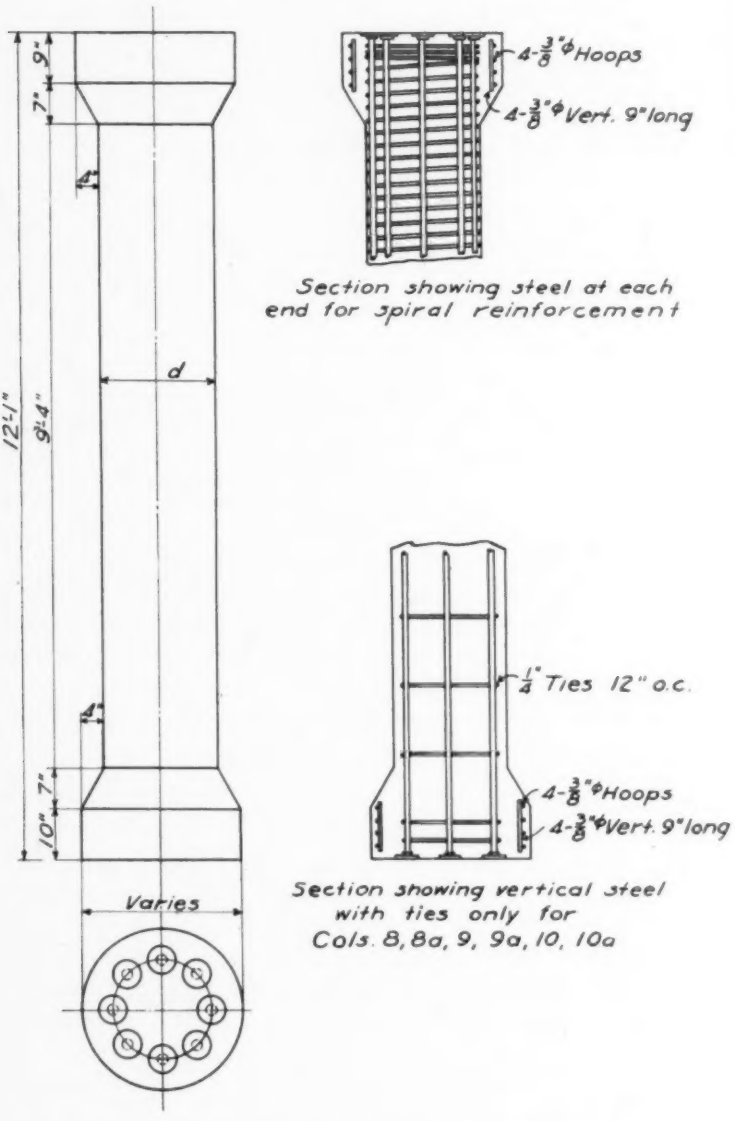


FIG. 1.—GENERAL DIMENSIONS AND DETAILS OF REINFORCED CONCRETE COLUMNS.

The amount passing a No. 100 sieve averaged 95.2 per cent for the three samples and that passing a No. 200 sieve averaged

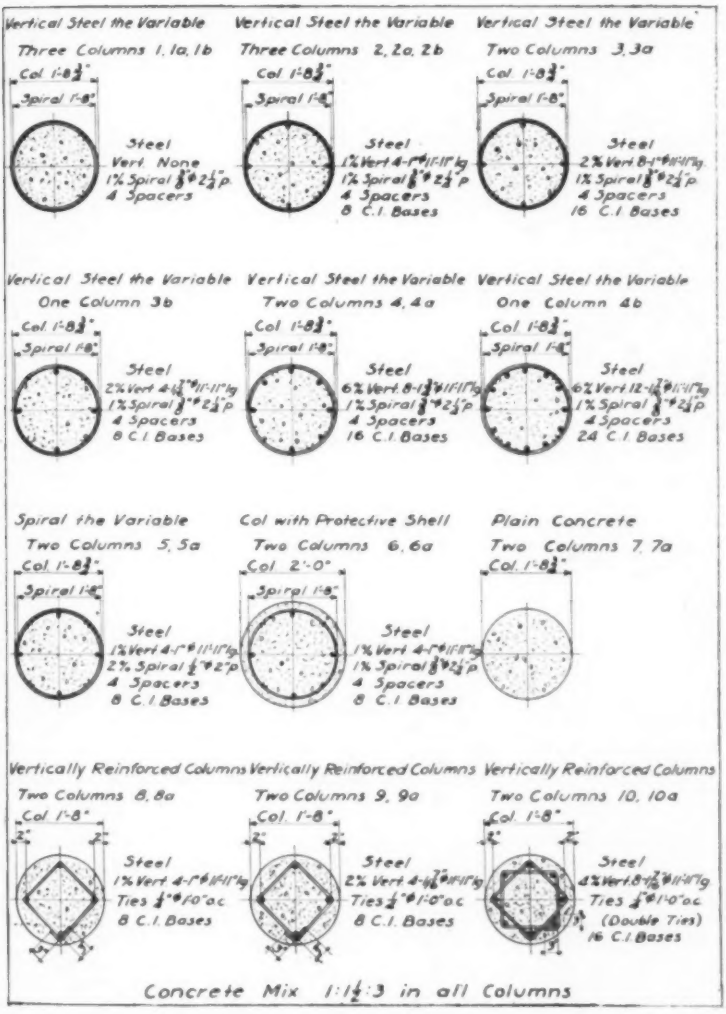


FIG. 2.—SECTIONS AND DETAILS OF TEST COLUMNS.

79.9 per cent. Table II gives the average tension tests for the three samples. In all cases the cement met the requirements of

the standard specifications of the American Society for Testing Materials (Standard No. 1 of the Institute).

Sand and Gravel.—The sand and gravel were taken from material delivered for use in the warehouse construction and were of good quality. The sand was siliceous, clean, hard and gritty. The stone contained some pebbles of feldspar and mica schist, but it was well graded and clean. The average results of the mechanical analysis of the sand and gravel are given in Tables III and IV.

The sand contained 35.5 per cent voids and had a specific gravity of 2.66. The ratio of the strength of these briquettes to

TABLE II.—TENSION TESTS OF CEMENT.

Each value for samples No. 1 and No. 3 is the average of five tests. For sample No. 2 each value is the average of three tests. Standard Ottawa sand was used in the 1:3 briquettes. The results are expressed in pounds per square inch.

Sample No.	Neat.			1-3.	
	1 day.	7 days.	28 days.	7 days.	28 days.
1	382	612	769	286	370
2	373	665	728	280	365
3	383	653	688	292	316
Average	379	643	728	286	350

those made with standard Ottawa sand was as follows: at age of 3 days, 1.51; at 7 days, 1.12; at 28 days, 0.98.

The gravel contained 34.4 per cent voids and had a specific gravity of 2.65.

Reinforcement.—The spiral reinforcement was bright Bessemer wire. During the time of winding the spirals, coupons from 20 to 24 in. long were clipped for testing. Five coupons of each size ($\frac{1}{2}$ in. and $\frac{3}{8}$ in.) were tested. Table V gives a summary of the results of the tension tests. Fig. 3 gives the average stress deformation curves for the tests made at the University of Illinois. The elastic limit reported in the University of Illinois tests is Johnson's elastic limit. It is seen from Fig. 3 that at this stress there is considerable deviation from proportionality of stress to strain.

For the longitudinal reinforcement, three sizes of bar were used, 1 in., $1\frac{7}{16}$ in., and $1\frac{3}{4}$ in. The bars were milled square on the ends to a length of 11 ft. 11 in. Four of the 1-in. bars were left over and these were used for physical tests.

Table V gives a summary of these tests and Fig. 4 gives a stress-deformation diagram for a tension test and a compression test of the longitudinal reinforcement. Compression tests were

TABLE III.—MECHANICAL ANALYSIS OF SAND.

Sieve No.	Separation Size, inches.	Per cent Passing, Average of Two Samples.
—	0.5	100.0
—	0.25	99.7
10	0.073	91.0
20	0.034	68.8
30	0.022	45.9
40	0.015	28.9
50	0.011	15.0
80	0.007	5.2
100	0.0045	3.5
200	0.00275	1.8

TABLE IV.—MECHANICAL ANALYSIS OF GRAVEL.

Size of Round Opening, inches.	Per cent Passing, Average of Two Samples.
$1\frac{1}{2}$	100.0
$1\frac{1}{4}$	99.8
1	96.9
$\frac{3}{4}$	83.0
$\frac{1}{2}$	55.7
$\frac{1}{4}$	14.9
$\frac{1}{8}$	2.8

made on three of the coupons. Table VI gives the chemical analysis of the longitudinal reinforcement.

IV. MAKING, TRANSPORTATION AND STORAGE.

In preparation for the work of making the columns, a 4-in. slab of concrete was built on the ground in a space in the warehouse of the Baltimore and Ohio Railroad in New York, which had been reserved for railroad tracks and which was inclosed on all but the south side. Care was taken to make this slab as nearly flat as possible. The column forms were set up on templates

of heavy paper on which were glued the cast iron base plates shown in Fig. 5, that located the longitudinal reinforcing bars. Similar cast iron plates were placed on top of the longitudinal reinforcement. The reinforcement was also wired to the forms.

TABLE V.—TENSION TESTS OF REINFORCEMENT.

Sample No.	Diameter, in.	Turner Construction Co. Tests.				University of Illinois Tests.			
		Elastic Limit, lb. per sq. in.	Ultimate Strength, lb. per sq. in.	Elongation in 8 in., per cent.	Reduction of Area, per cent.	Elastic Limit ‡ lb. per sq. in.	Ultimate Strength, lb. per sq. in.	Elongation in 8 in., per cent.	Reduction of Area, per cent.
1	¾	.....	84,600	...	55.5	55,500	79,500	5.0	60.0
2	"	....	82,500	4.0	55.5	70,000	88,500	5.0	53.0
3	"	.....	82,800	4.0	55.5	61,000	85,900	3.8	54.0
4	"	.....	84,100	...	54.7	66,000	88,000	...	53.0
5	"	60,000	85,900	3.6	48.1	.....	.....	...	....
6	"	62,000	83,500	3.8	48.1	.....	.....	...	....
Average	.....	61,000	83,900	3.8	52.9	63.100	85,500	4.6	55.0
1	½	.....	86,800	...	44.5	58,500	89,600	...	53.0
2	"	.....	86,400	4.3	44.5	70,500	88,000	...	51.0
3	"	.....	85,100	...	39.6	61,500	87.100	5.0	51.0
4	"	58,000	86,500	3.4	43.0	60,500	88,200	5.0	53.0
5	"	55,000	86,600	4.1	48.5	.....	.....	...	....
Average	.....	56,500	86,300	3.9	44.0	62,800	88,200	5.0	52.0
1	1	34,000	56,180	35.5	64.0	35,200	57,500	32.5	47.0
2	"	33,200	55,900	30.5	48.1	36,200	57,200	32.5	47.0
3	"	33,900	56,860	32.4	48.3	34,500†	.....	....	....
4	"	33,900	56,650	33.5	52.9	35,000†	.....	....	....
5	"	33,300	56.510	34.3	51.5	37,000†	.....	....	....
Average	.....	33,660	56,420	33.2	53.0	35,700	57,350	32.5	47.0
*	1	37,680	58,570	29.7	53.8	.....	.....	....	....
*	1 7/16	37,430	59,400	30.7	51.1	.....	.....	....	....

* Carnegie Steel Co. tests.
† Compression tests, not included in the average.
‡ Johnson's Elastic Limit.

TABLE VI.—CHEMICAL ANALYSIS OF REINFORCING BARS.

Size of Bar, inches.	Per cent.				
	Carbon.	Manganese.	Phosphorus.	Sulphur.	Iron.
1	0.18	0.41	0.012	0.033	99.375
1 7/16	0.19	0.46	0.019	0.035	99.196

The forms were of wood, tight and well-made. The inside of the forms was brushed with oil. A quarter-section of the form at the base was left open until after the reinforcement was in place and inspected. Care was taken to see that the spacers

on the spiral reinforcement were secured in a vertical position. After sweeping out the loose pieces of tie wire and other refuse within the form, the opening was closed.

The plant for mixing and placing the concrete was the same as that used in the construction of the building and care was taken to obtain uniformity of conditions throughout. The workmen employed were those who had worked upon the construction

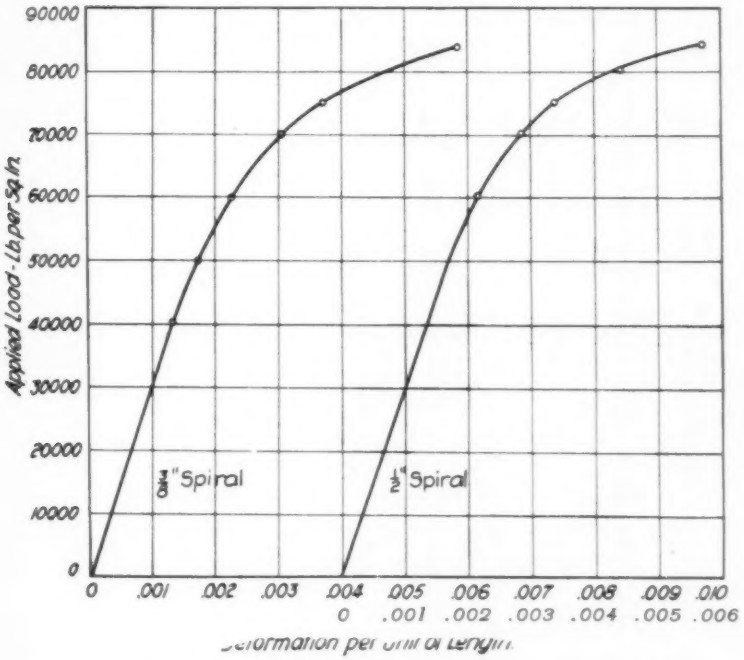


FIG. 3.—STRESS-DEFORMATION DIAGRAM FOR SPIRAL REINFORCEMENT. (AVERAGE OF 4 TESTS FOR EACH SIZE.)

of the building. A batch mixer of one-half yard capacity was used. Aggregates and mixing water were not heated. Each batch consisted of four bags of cement (which was taken as 4 cu. ft.), 6 cu. ft. of sand and 12 cu. ft. of gravel by loose measure. The sand and gravel were measured in two-wheel carts of 6 cu. ft. capacity. Before using, these carts were calibrated with a cubic foot box, and each load was stricken off as loaded. About seven pailfuls of water (200 lb.) were used for each batch. The mixer

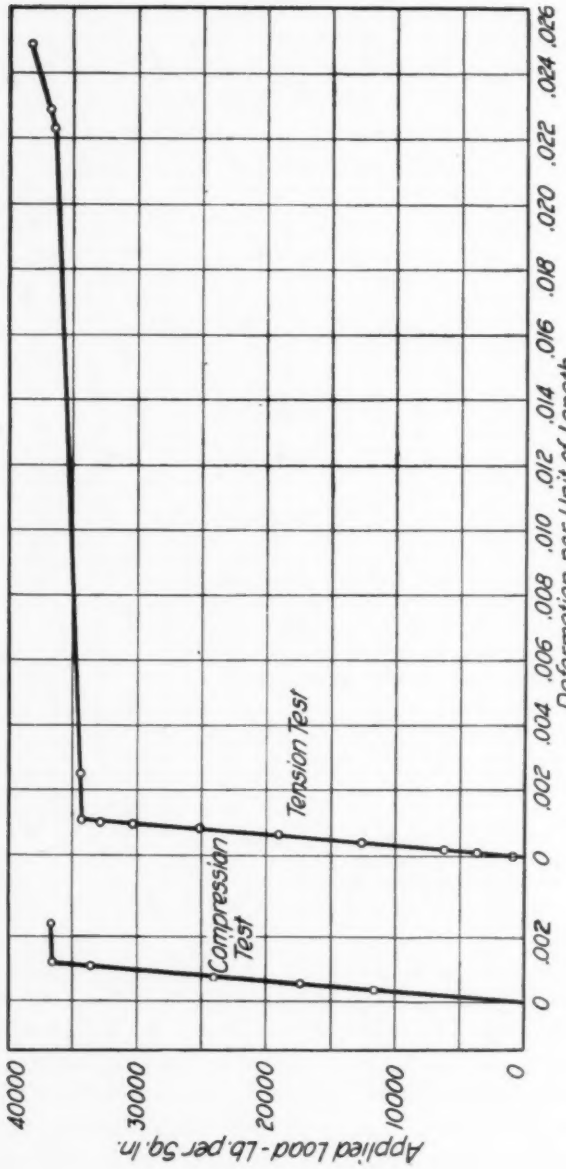


FIG. 4.—STRESS-DEFORMATION DIAGRAM FOR LONGITUDINAL REINFORCEMENT.

was run at 22 revolutions per minute. In general, each batch was mixed three minutes or more, the period depending upon the time taken to pour a batch into the column form. In general, each column required about two batches of concrete. After the concrete was poured into the forms, it was constantly poled in the manner used by the same gang of workmen in filling the concrete columns in the construction of the building. The column forms

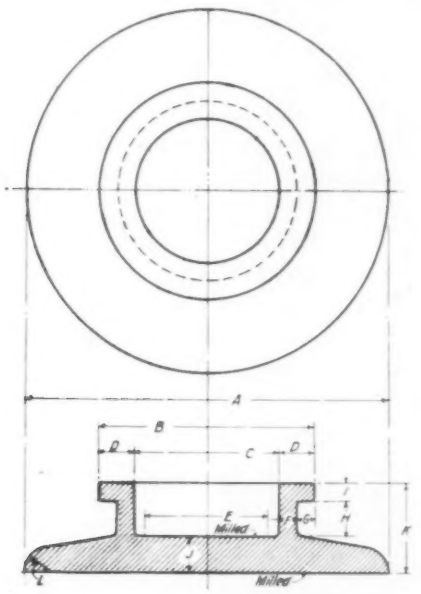


FIG. 5.—DETAIL OF CAST IRON BEARING PLATES.

A—5 "—3½"	G— ¼"— ½"
B—3 "—2½"	H— ½"— ¾"
C—2 "—1½"	I— ½"— ¾"
D— ½"— ½"	J— ½"— ½" (exactly.)
E—1½"—1"	K—1¼"—1½"
F— ¼"— ½"	L— ¼"— ⅜"

were used three times. They were removed after three days, except for the eight columns last filled, which were not removed until just before shipment.

Two test cylinders were made from separate batches of the concrete put into each column. The test cylinders were kept in the inclosure with the columns. They were wet down once in an effort to compensate for drying out more rapidly than the columns.

The temperature of the air on the days the columns were poured was as follows: November 19, 59 deg. F.; November 22, 56 deg. F.; November 26, 46 deg. F. In order to protect the columns from the freezing weather which came before they were shipped, they were surrounded and covered with tarpaulins and a salamander was kept burning in the inclosure to maintain a temperature of 50 deg. F., except during the time between December 1 and December 23. From December 1 to December 23 and also from January 3 to January 19 while the columns were being loaded and shipped to Pittsburgh the columns and cylinders were exposed to moderate winter weather.

The columns and cylinders were shipped to Pittsburgh on two flat cars. Care was taken to block the columns so that they could not shift in transit. They were seven days en route, and were unloaded without apparent damage. The columns were then stored on the testing floor of the Bureau of Standards laboratory. As the temperature of this room was frequently very low, especially at night, the columns were grouped close together and entirely inclosed by canvas. Gas was burned within the inclosure for several weeks, and a moderate temperature maintained. Taken in all, it is believed that the conditions for hardening in the 115 days which elapsed between making and testing were less favorable than may be expected with a temperature of 65 to 70 deg. F. maintained for 90 days.

V. PREPARATION OF COLUMNS FOR TESTING.

The preparation of the columns for testing consisted of cutting into the concrete to expose the reinforcement to give opportunity for measuring the deformation in the reinforcement, setting steel plugs firmly in holes drilled in the concrete in the proper positions for the measurement of deformation of the concrete, and drilling the gage holes.

For cutting the concrete, a tool driven by compressed air was used. The procedure in setting the plugs was as follows: The concrete in the vicinity of the gage holes was removed flush with the outer surface of the spiral. A steel plug was set flush with the outer surface of the longitudinal or spiral reinforcement. The method used is indicated in Fig. 6

Fig. 7 shows the location of the gage lines upon the developed surface of a column and their numbering. There were slight

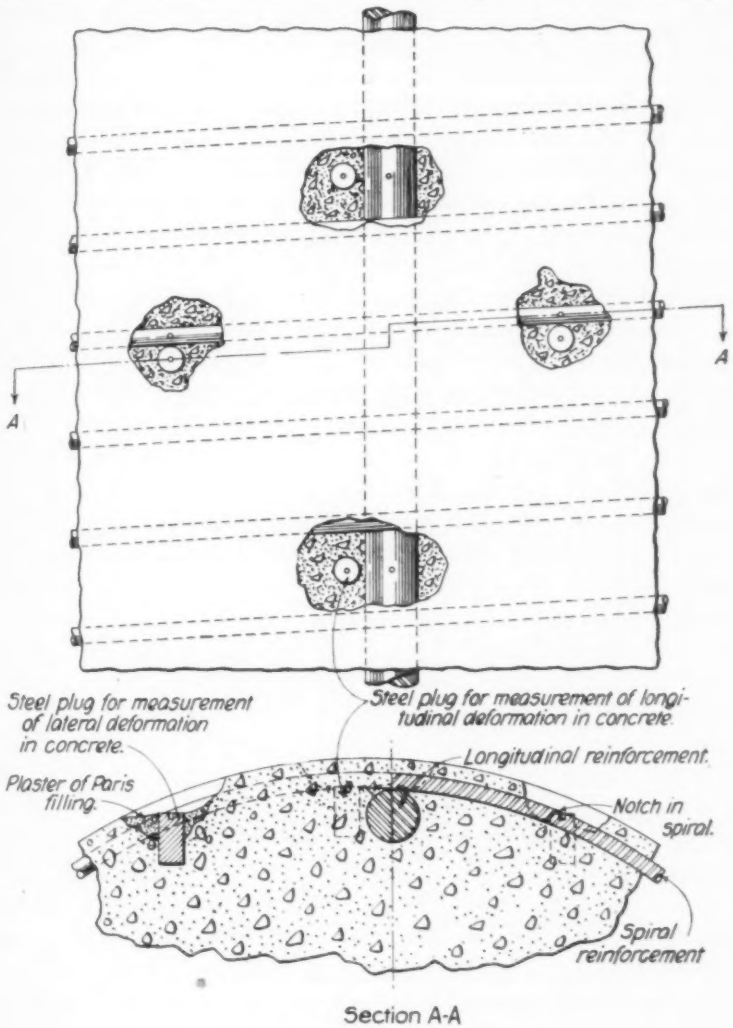


FIG. 6.—DETAIL OF LAYOUT OF GAGE LINES.

variations from this layout in individual columns. The number of gage lines varied from 40 to 112, but in most of the columns there were about 80.

VI. TESTING.

The columns were tested in the 10,000,000-lb. testing machine of the Bureau of Standards laboratory at Pittsburgh, Pa. In this testing machine the load is applied by means of a hydraulic piston

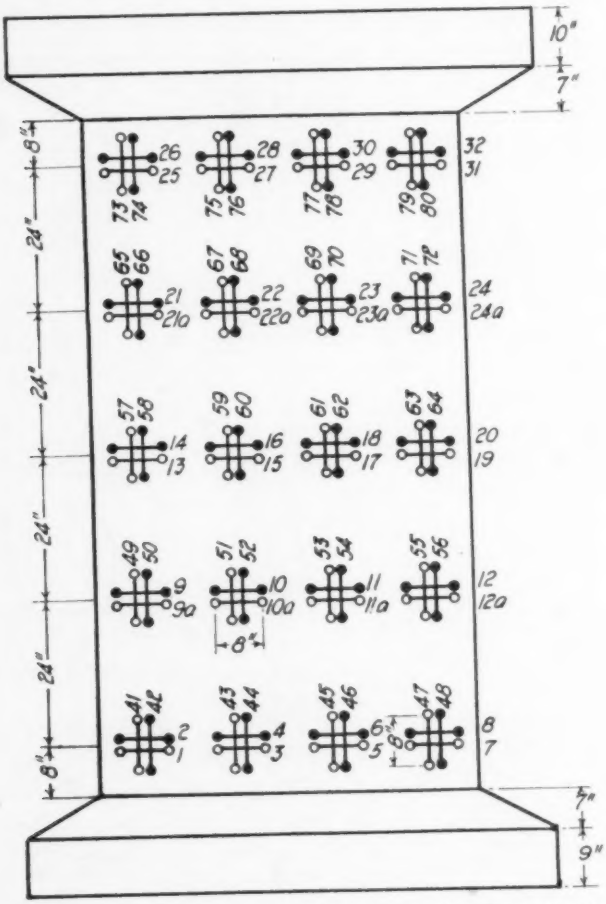


FIG. 7.—DEVELOPED SURFACE OF COLUMN, SHOWING LOCATION OF GAGE LINES.

which is actuated by a hydraulic pump. The load is weighed through a system of levers which are connected with the piston of a smaller hydraulic cylinder which communicates with the larger cylinder.

The columns were placed in position for testing after the gage lines had been prepared and numbered. A layer of plaster of Paris was used on the top and bottom surfaces of the columns. In placing the columns in the machine care was taken to have the upper surfaces of the column parallel with the upper platen of the testing machine and placed in such a way as to make external eccentricity of load a minimum. Any lateral thrust due to lack of uniformity in the action of the columns themselves was resisted by the four large main screws. The weighing beam of the testing machine was counterbalanced with the pump operating slowly. The pump was then stopped, the motor operating the top head was started, and the top head was brought down on the soft plaster of Paris at the top of the column until plaster of Paris was forced out on all sides. An initial load of from 50,000 to 100,000 lb. was applied. The scale reading of the initial load was taken with the oil pump operating, and the piston moving very slowly.

The practice of the laboratory, under the instructions of the maker of the testing machine, is to balance the weighing beams while the pump is working, the purpose being to reduce the piston friction of rest to that of motion. The loads were read 2 minutes later and generally also 20 or more minutes later, but these readings were taken with the pump at rest and hence the conditions of the testing machine were somewhat different from those for which the first reading was taken.

After the column had been placed in the testing machine and the initial load had been applied, the initial strain-gage readings were taken. Berry strain-gages of the form in use at the University of Illinois were used.* At least two sets of initial readings were taken in all cases to serve as a check on each other. If the two sets showed serious disagreement in any gage line, a third set of readings was taken on these gage lines.

A full set of strain-gage readings was taken at each load. The time of taking a set of observations was twenty minutes or more. From 6 to 15 sets of readings, corresponding to an equal number of loads, were taken in the test of a column, making from 250 to 1300 observations in all.

* For a description of this instrument and its use, see "The Testing of Reinforced Concrete Buildings under Load" in *Proceedings*, Vol. VIII, 1912, p. 168.
See also "The Use of the Strain Gage in the Testing of Materials," by W. A. Slater and H. F. Moore, Vol. XIII, *Proceedings*, Am. Soc. for Test. Mats., and "The Testing of Reinforced Concrete Buildings under Load," by A. N. Talbot and W. A. Slater, Bulletin 64, Univ. of Ill. Eng. Experiment Station.

The amount of load reported is the load indicated at the instant before stopping the pump of the testing machine. In any material having some plasticity a test piece under dead load will continue to shorten after the initial application of the load, or the test piece will be carrying less load if it is held at a constant length as is the case in a test of this kind. When a hooped column is loaded nearly to its ultimate capacity, it acts with some plasticity. To enable the extensometer readings to be taken under the same conditions over the whole column, the pump was kept at rest throughout the time of observation, but readings of the indicated load were taken at two minutes after the first reading and again at 20 minutes or more later. These readings are lower than the first reading, in some cases as much as 10 per cent less. How much of this decrease is due to the yielding of the column and how much to different conditions of weighing the load is not known. However, it is apparent that the real maximum load on a column under a rational interpretation of the application of the load would be somewhat less than the values here reported. It should be also added that the statement made in the discussion that the load held for a time, such as one minute, means that there was no great loss of load beyond the usual dropping of the indicated load, as found in the tests generally.

After the extensometer readings were taken at the maximum load, the pump of the testing machine was again put into operation in order to determine the manner of the final failure and to find how the column would act under continued deformation. The photographs which were taken show the columns as they appeared after having the deformation carried well beyond that produced by the maximum load. The spalling, which in the photographs appears to have removed much of the concrete outside the spiral, was much less pronounced at the maximum load.

VII. DATA OF TESTS.

The compression strength of the concrete is given in Table VII.

The dates of making and testing the columns and cylinders are given in Table VIII. Table IX gives the ultimate loads carried by the columns and by the concrete cylinders, as well as general data of the test columns, while Table X gives general data

for the spirally reinforced columns. Fig. 8 gives a typical stress deformation diagram for the concrete cylinders. These cylinders were made with column 1. Stress-deformation diagrams are

TABLE VII.—COMPRESSIVE STRENGTH OF CONCRETE.

Average age, days.	Compressive Strength, lb. per sq. in.	Remarks.
7	872	Average of 2, 8 x 16 in. cylinders. Air cured.
28	1966	Average of 6, 8 x 16 in. cylinders. Air cured.
116	2943	Average of 48, 8 x 16 in. cylinders. Air cured.

TABLE VIII.—DATES OF MAKING AND TESTING COLUMNS AND CYLINDERS.

Column No.	Date Made, 1913.	Date Tested, 1914.	Age, days.	Companion Cylinders.	
				Date Tested.	Age, days.
1	November 19	February 27 and 28	101	March 3	104
1a	" 22	March 11	109	" 11 and 13	109, 111
1b	" 26	" 25	119	" 25	119
2	November 19	March 3	104	March 2	103
2a	" 22	" 16	114	" 17	115
2b	" 26	" 23	117	" 23	117
3	November 19	March 4	105	March 4	105
3a	" 22	" 17	115	" 17	115
3b	" 26	" 26	120	" 26	120
4	November 19	March 5	106	March 5	106
4a	" 22	" 18 and 19	117	" 18	116
4b	" 26	" 28	122	" 27 and 28	121, 122
5	November 19	March 6	107	March 6	107
5a	" 22	" 20	118	" 20	118
6	November 19	March 12	113	March 13	114
6a	" 22	" 19	117	" 20 and 23	118, 121
7	November 26	March 27	121	March 27	121
7a	" 26	" 30	124	" 31	125
8	November 19	March 10	111	March 9 and 11	110, 112
8a	" 22	" 23	121	" 23	121
9	November 19	March 14	115	March 9 and 14	110, 115
9a	" 22	" 24	122	" 24	122
10	November 26	March 31	125	March 31	125
10a	" 26	April 1	126	April 1	126

given in Figs. 25 to 48, plotted from the data taken in the tests. Elongation is plotted as positive and shortening as negative. The dotted lines represent measurements of the gage lines on the concrete and the full lines represent the measurements of

TABLE IX.—GENERAL DATA OF TESTS.

In columns with ½-in. spiral the pitch was 2 in.; in columns with ⅜-in. spiral the pitch was 2¼ in.

Column No.	Section.		Spiral Reinforcement.			Longitudinal Reinforcement.		Maximum Load, lb.	Unit Load, lb. per sq. in.	Average Strength of 8 x 16 in. Cylinders, lb. per sq. in.
	Diameter, in.	Area, sq. in.	Kind.	Per cent.	Diameter, in., c. to c.	Kind.	Per cent.			
1	20.84	341	½-in.	0.94	19.95	none	0.00	1,621,500	4760	2868
1a	20.82	340	do.	0.94	19.97	do.	0.00	1,551,000	4562	3083
1b	20.86	341	do.	0.94	19.93	do.	0.00	1,774,000	5210	2885
Average									**4845**	
2	20.98	345	½-in.	0.94	18.69	4 1-in. rods	0.91	1,626,000	4710	3393
2a	21.02	347	do.	0.93	19.68	do.	0.91	1,700,000	4900	2773
2b	20.94	344	do.	0.94	19.82	do.	0.91	1,766,000	5140	2860
Average									**4920**	
3	20.95	344	½-in.	0.94	19.37	8 1-in. rods	1.83	1,737,500	5050	3185
3a	20.96	344	do.	0.94	19.85	do.	1.83	1,647,000	4785	2575
3b	20.94	344	do.	0.94	19.91	4 1⅞-in rods	1.89	2,017,000	5863	2675
Average									**5235**	
4	20.66	335	½-in.	0.95	19.53	8 1¾-in. rods	5.75	2,069,000	6180	3095
4a	20.88	342	do.	0.94	19.76	do.	5.63	1,968,000	5750	2670
4b	20.85	341	do.	0.94	19.74	12 1⅞-in. rods	5.71	2,165,000	6350	2775
Average									**6093**	
5	20.87	342	½-in.	1.88	19.57	4 1-in. rods	0.92	2,060,000	6025	3035
5a	20.92	344	do.	1.88	19.73	do.	0.91	2,233,000	6491	2883
Average									**6258**	
6	24.09	455	½-in.	....	.....	4 1-in. rods	....	1,740,000	3825	3130
6a	24.23	462	do.	....	.....	do.	....	1,720,000	3725	2768
Average									**3775**	
7	20.87	342	none	0.00	.....	none	0.00	1,022,000	2990	2775
7a	20.96	344	do.	0.00	.....	do.	0.00	860,000	2500	2518
Average									**2745**	
8	20.04	315	....	....	.....	4 1-in. rods	1.00	1,040,000	3300	3308
8a	20.31	324	....	....	.....	do.	0.97	1,360,000	4200	3098
Average									**3750**	
9	20.13	318	....	....	.....	4 1⅞-in. rods	2.04	1,150,000	3620	3458
9a	20.21	321	....	....	.....	do.	2.02	1,206,000	3760	3055
Average									**3690**	
10	20.14	318	....	....	.....	8 1⅞-in. rods	4.08	1,451,000	4565	2835
10a	20.16	319	....	....	.....	do.	4.07	1,436,000	4500	2940
Average									**4535**	

TABLE X.—GENERAL DATA FOR THE SPIRAL REINFORCED COLUMNS.

Column No.	Section, Center to Center, of Spirals.		Spiral Reinforcement.			Longitudinal Reinforcement.		Maximum Load, lbs.	Unit Ultimate Load, lb. per sq. in. Net Section.	Average Strength, 8 x 16 in. Cylinders, lb. per sq. in.
	Diameter, ins.	Net Area, sq. ins.	Size Wire.	Pitch.	Per Cent Net Section.	Amount.	Per Cent Net Section.			
1	19.96	312.9	¼-in.	2½-in.	0.98	None	....	1 621 500	5 180	2 868
1A	19.98	313.6	do.	do.	0.98	do.	....	1 551 000	4 945	3 083
1B	19.94	312.3	do.	do.	0.98	do.	....	1 774 000	5 680	2 885
Average									5 270	2 945
2	18.72	275.2	¼-in.	2½-in.	1.05	4 1-in. rods	1.14	1 626 000	5 910	3 393
2A	19.71	305.1	do.	do.	1.00	do.	1.03	1 700 000	5 570	2 773
2B	19.81	308.2	do.	do.	.99	do.	1.02	1 766 000	5 730	2 860
Average									5 735	3 008
3	19.35	294.1	¼-in.	2½-in.	1.01	8 1-in. rods	2.14	1 737 500	5 910	3 185
3A	19.85	309.4	do.	do.	.99	do.	2.03	1 647 000	5 325	2 575
3B	19.92	311.6	do.	do.	.99	4 $1\frac{7}{16}$-in. rods	2.08	2 017 000	6 475	2 675
Average									5 900	2 812
4	19.52	299.2	¼-in.	2½-in.	1.01	8 1¼-in. rods	6.43	2 069 000	6 915	3 095
4A	19.77	307.0	do.	do.	.99	do.	6.27	1 968 000	6 410	2 670
4B	19.75	306.3	do.	do.	.99	12 $1\frac{7}{16}$-in. rods	6.36	2 165 000	7 070	2 775
Average									6 800	2 847
5	19.56	300.5	¼-in.	2-in.	2.01	4 1-in. rods	1.05	2 060 000	6 855	3 035
5A	19.73	305.7	do.	do.	1.99	do.	1.03	2 233 000	7 305	2 883
Average									7 080	2 959
6	19.76	306.7	¼-in.	2½-in.	.99	4 1-in. rods	1.02	1 740 000	5 675	3 130
6A	19.76	306.7	do.	do.	.99	do.	1.02	1 720 000	5 610	2 768
Average									5 640	2 949

Table X differs from Table IX in the basis of computing the unit stress for the columns having spiral reinforcement. In Table IX the unit stress was determined by dividing the total maximum load on the columns by the corresponding gross cross-sectional area. In Table X, however, the unit stress was computed on the basis of the net section within the center line of the spirals. Table X, therefore, is based on the assumption that the shell of concrete outside of the spiral is negligible as a factor in the strength of spirally reinforced columns at ultimate loads.

the gage lines on the reinforcement. Fig. 9 is an enlarged stress deformation diagram giving the average measurements for the central plane shown in Fig. 33. Fig. 10 gives the distribution of deformations over the length of one column, 2b. This is a dia-

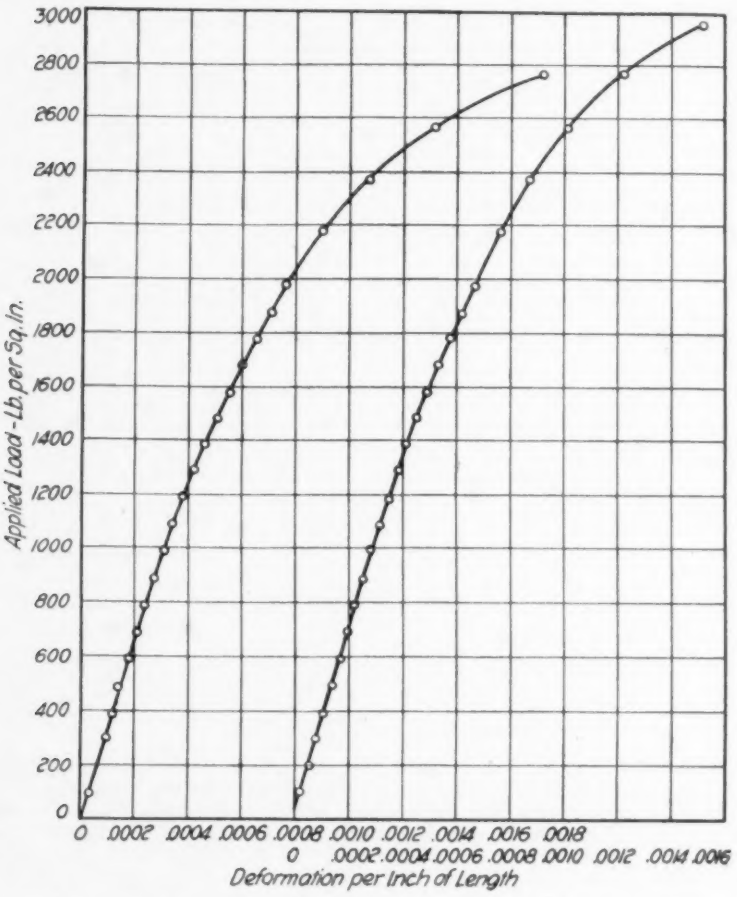


FIG. 8.—TYPICAL STRESS-DEFORMATION DIAGRAM OF CONCRETE CYLINDERS

grammatic representation of the longitudinal compressive deformations and the lateral tensile deformations in the concrete. The measurements were taken at five planes, located 8 in. from the top and bottom and at three intermediate levels as shown on the

diagram. On the left are shown the measurements on one face and on the right those on the opposite face. In Figs. 11 and 12, the loads have been plotted according to the amount of longitudinal and spiral reinforcement.

VIII. PLAIN CONCRETE COLUMNS.

The two plain concrete columns showed similar characteristics in the tests. No cracks or other signs of failure were visible until

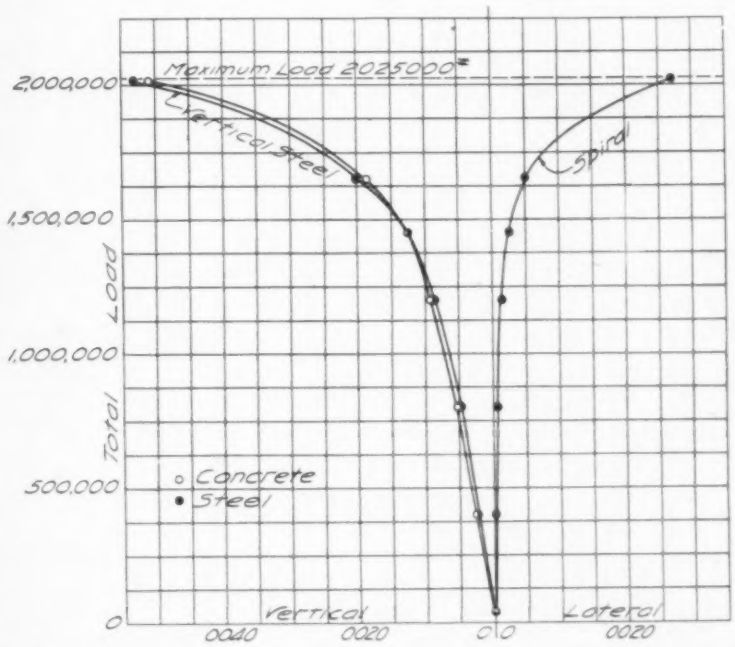


FIG. 9. ENLARGED STRESS DEFORMATION DIAGRAM GIVING THE AVERAGE MEASUREMENT FOR THE CENTRAL PLANE SHOWN IN FIG. 33.

the maximum load was reached, except that in Column 7a a small longitudinal crack was seen in the shaft just below the head at a load of 750,000 lb. (2190 lb. per sq. in.). At the maximum load, longitudinal cracks developed all around the shaft of the column below the top of the head and extended down over the upper quarter of the length of the shaft. The load at once fell off 20

per cent or more, and with renewed movement of the pump, the load dropped still further. Column 7a and its test cylinders gave results which are lower than the average of all the test cylinders,

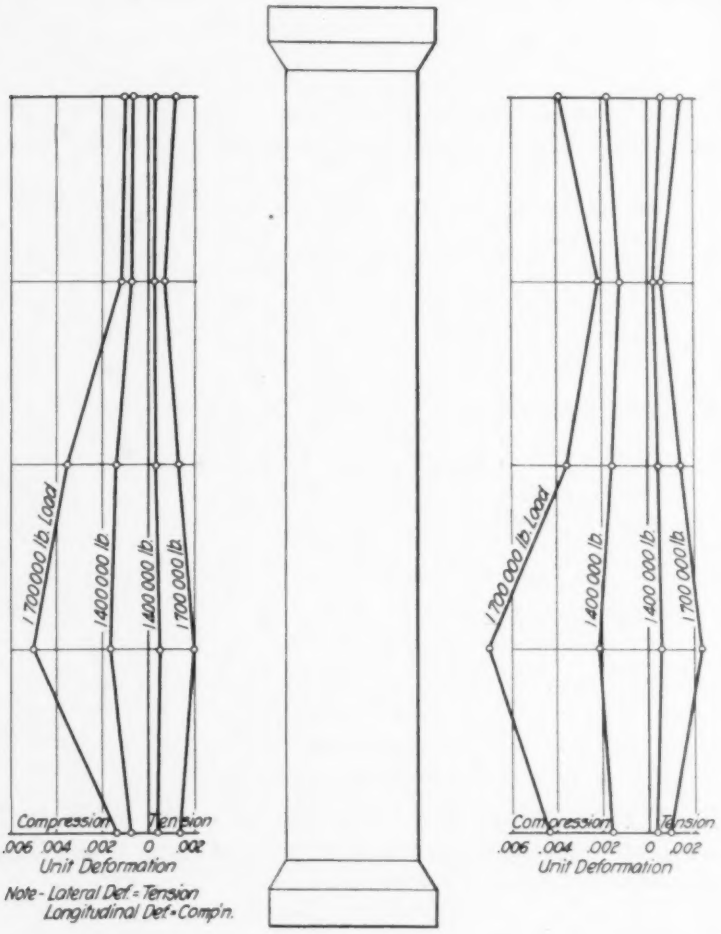


FIG. 10.—DISTRIBUTION OF LONGITUDINAL AND LATERAL DEFORMATION OVER LENGTH OF COLUMN FOR TWO LOADS. DIAGRAMS ON THE LEFT AND RIGHT ARE FOR OPPOSITE FACES

and the action of 7a in the test indicates that the strength of this column was below what would be expected with concrete having the average quality of that in the other columns.

IX. COLUMNS WITH LONGITUDINAL REINFORCEMENT AND NO SPIRAL REINFORCEMENT.

The series having longitudinal reinforcement and no spiral reinforcement includes two columns with 1 per cent of reinforcement, two columns with 2 per cent of reinforcement and two columns with about 4 per cent of reinforcement. Ties $\frac{1}{4}$ in. in diameter were placed 12 in. apart. In the tests there was no

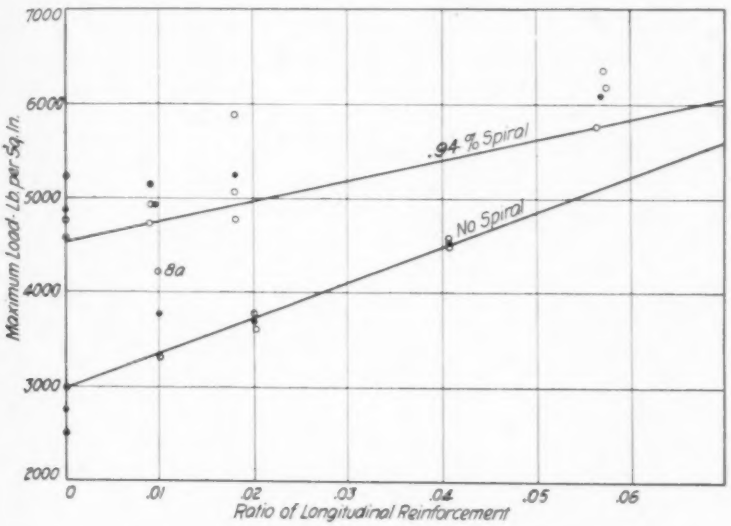


FIG. 11.—ULTIMATE LOADS—LONGITUDINAL REINFORCEMENT VARIABLE.

marked sign of distress before the maximum load was reached. In some cases no crack was visible below the maximum load. In other columns a minute crack was found at 85 to 90 per cent of the maximum load. Scaling of the longitudinal reinforcement at the gage holes where the bars were visible was seen at loads below the maximum. In general the load fell off considerably a half-minute or so after the maximum load was applied.

The maximum loads carried by the columns are plotted in Fig. 11. The load taken by the longitudinal reinforcement is about 40,000 lb. per sq. in. The line shown in the diagram is expressed by the equation

$$f = 3000\ (1-p) + 40000p$$

where f is the average unit load over the whole column section, and p is the ratio of the area of longitudinal reinforcement to the area of the column. This formula in effect assumes that at the maximum load the concrete carries 3000 lb. per sq. in. and the reinforcement 40,000 lb. per sq. in., making $n = 13\frac{1}{3}$.

X. COLUMNS WITH SPIRAL REINFORCEMENT AND WITH AND WITHOUT LONGITUDINAL REINFORCEMENT.

The columns having 1 per cent of spiral reinforcement included columns with no longitudinal reinforcement and columns with nominally 1, 2 and 6 per cent of longitudinal reinforcement.

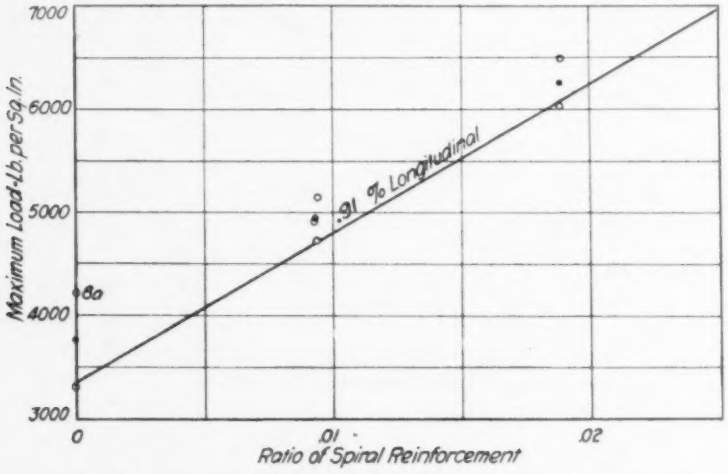


FIG. 12.—ULTIMATE LOADS; SPIRAL REINFORCEMENT VARIABLE.

Two columns contained 2 per cent of spiral reinforcement and 1 per cent of longitudinal reinforcement. The four spacers used to hold the spiral reinforcement in place had a sectional area equivalent to about 0.2 per cent of the column section, but this metal has not been considered in computing the percentages of longitudinal reinforcement.

The stress-deformation curves for columns with spiral reinforcement are shown in Figs. 25 to 40 inclusive, and for unhooped columns the curves are shown in Figs. 43 to 48 inclusive. Both longitudinal and lateral deformations are plotted.

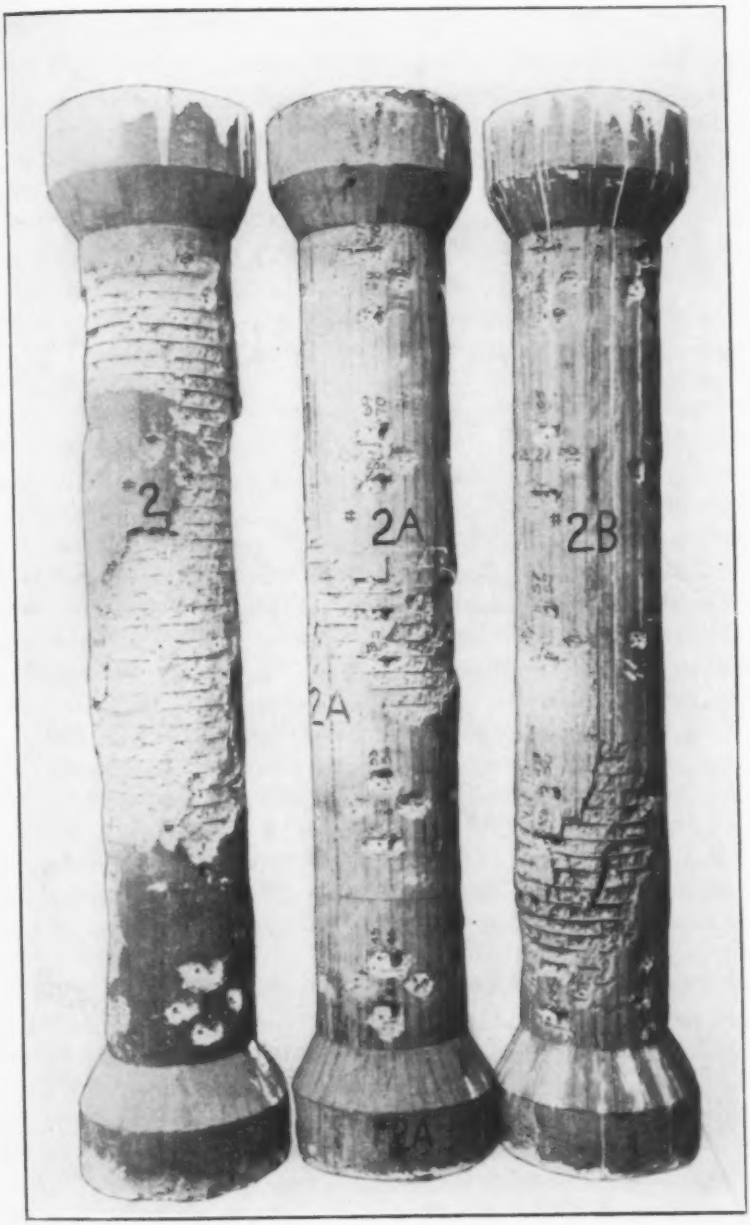


FIG. 13.—COLUMNS 2, 2A AND 2B AFTER FAILURE.

The action of the hooped columns during the early part of the tests was similar to that of the unhooped columns. At a load near the maximum load which was carried by the unhooped columns having an equal amount of longitudinal reinforcement, the observed stress in the spiral was, say from 6000 to 8000 lb. per sq. in., an amount which may be expected to correspond to the lateral expansion of the columns under slight restraint. The effect of this restraint at these loads was slight, perhaps equivalent to 150 lb. per sq. in. vertical pressure in the columns with 1 per cent spiral. The scaling of the longitudinal reinforcement where exposed at the gage holes was observed at loads similar to those observed for the unhooped columns.

Beyond the load which corresponds to the maximum load of an unhooped column the column shortened more rapidly with increase of load, the change in stiffness being very noticeable, and the stress in the spiral increased very rapidly. The decreasing stiffness of the column was very marked, and at loads near the maximum the ratio of the added load to the added longitudinal shortening was for columns with 1 per cent of spiral, say $\frac{1}{15}$ of that found at loads below the maximum load of an unhooped column. With continued increase of load the stretch of the spiral increased and beyond the yield point of the spiral reinforcement this stretch increased very rapidly relatively to the increment of load. As the maximum load was approached, the spiral stress was no longer uniform around the circumference, and it varied at different parts of the length of the column. Finally at some point the maximum strength of the spiral reinforcement was reached or the action of the column became very far from uniform, and the maximum load which the column would carry was attained.

There were few signs of distress or of approaching failure until near the maximum load. There were no spacing bars outside of the spiral to buckle. There was little or no spalling of the concrete outside the spiral, such as has been found in column tests when the coating outside the spiral was relatively much thinner. Fine vertical cracks usually appeared somewhat before the maximum load. Spalling occurred in one or more parts of the column—generally in several—either a little in advance of the maximum load or at the maximum load. The maximum load was held for

some time, a minute or more, the pump operating during this time and the spiral reinforcement continuing to stretch. In many cases the spiral broke either at the maximum load, or at later and lower loads obtained by continuing to run the pump of the testing machine. The amount of shortening at the maximum load was generally about .004 in. per inch of length or more for 1 per cent spiral and .006 in. per inch for 2 per cent spiral.

In discussing the properties of the hooped columns, the question at once arises as to what part of the cross-section of the column is affected by the restraint of the spiral and whether the whole section acts to resist the load imposed on the column. The unit stress for any load will vary with the area of the cross-section used, and there will be a marked difference between the value obtained by using the core section and that found with the area of the entire cross-section of the column. The section which gives the proper interpretation of the action of the column is wanted in this study rather than the section which should be used in designing columns. Below the loads at which the spiral exerts an important effect, the conditions are similar to those of an unhooped column, the full cross-section should be used, and a proper interpretation will involve the use of unit stresses calculated on this basis. For loads which give marked stresses in the spiral it is obvious that all or nearly all of the area within the spiral will be fully restrained. It seems evident that an area outside the spiral may be acted upon similarly, the restraint being carried by the shearing and tensile resistance of the concrete. For the columns in question, the space entirely outside the spiral amounted generally to about $\frac{3}{8}$ in., that is, a space equal to the diameter of the spiral reinforcement for the 1 per cent spirals. These columns showed no spalling up to nearly the maximum load. Calculations and diagrams made by using the core area only gave results which are inconsistent among themselves and not satisfactory in the interpretation of the data. The area of the gross section may be somewhat greater than the area which is effectively restrained, but it cannot be far from the right one. The unit stresses so found are smaller than would be obtained otherwise, and the variation from the real effective section will partly make up for the possibility of having used for the ultimate loads values somewhat greater than may be warranted by the plastic quality

which occurs in a hooped column near its maximum load and when the spiral has been stressed beyond its yield point. It seems best, therefore, to use the gross area of the cross-section of the hooped columns. This should not be taken to have a bearing upon what section should be used in designing columns.

In Table IX the average unit loads given are based upon the gross section of the columns, while those in Table X are based on the net area. In Fig. 11 and Fig. 12 the plotted values are also based on the gross section.

From Fig. 11 it is apparent that the additional strength given by the longitudinal reinforcement is less in the case of columns with spiral reinforcement than in columns without spiral reinforcement. For loads beyond the resistance given in unhooped columns, it would seem that the longitudinal bars may offer little or no additional resistance, and that the lateral swelling of these bars may increase the stress produced in the spiral reinforcement without giving correspondingly increased strength to the column.

In any study of the results attention may well be called to the abnormally high strength of column 8a, which is inconsistent with the other tests. Except for this column, the results are fairly consistent among themselves. The lines drawn on Figs. 11 and 12 represent the equation

$$f = 3000\,(1-p) + 40000\,p + 160000\,(1-10p)q.$$

where f is the average unit stress over the whole column section, p is the ratio of the area of the longitudinal reinforcement to the area of the column section, and q is the ratio of the volume of the spiral reinforcement to the volume of the column. As the columns with 2 per cent spiral reinforcement contain only 1 per cent longitudinal reinforcement, there is nothing in the tests to show that this equation properly expresses the effect of longitudinal reinforcement in connection with 2 per cent spiral reinforcement. The formula, however, brings out a probable limitation on longitudinal reinforcement in hooped columns. Such a formula is limited by the range of the test variables, and it should be noted that the largest amount of spiral reinforcement in these tests is 2 per cent.

In discussing the results of the tests and in studying the diagrams in Figs. 11 and 12, it may be well to take into considera-

tion the area of the spacers as affecting the percentages of longitudinal reinforcement and the applicability of the formula.

The strength given by spiral reinforcement is larger than has been found in other tests which have been reported previously. In judging the results of the tests it should be borne in mind that these are short columns (less than 6 diameters between heads), that the spiral reinforcement was well made and properly placed, that the columns were straight, and that the loads were applied in such a way as to produce little bending moment in the columns. It has been shown by tests that irregularity of pitch and inclination of spiral produce the same effect as crookedness and may reduce the column strength.

In the discussion above, maximum loads are used and the values given are the ultimate strengths of the columns. A study of the load-stress diagrams made up from the data indicates that for any column the additional load due to the hooping is nearly proportional to the stress in the spiral reinforcement. This holds up to a stress of, say 85 per cent of the ultimate strength of the spiral reinforcement, when a marked variation in the stresses develops in different parts of a section. Rapid increase of stress at some point soon follows.

XI. COLUMNS WITH FIREPROOFING LAYER.

The two columns with a thickness of about 2 in. of concrete outside the spiral were tested to determine the action of the fireproofing layer or protective shell under high loads. The action of the column through the first part of the test was similar to that of an unhooped column. At the maximum load the longitudinal deformations were about those of an unhooped column at its maximum load, somewhat less than the usual amount at most points of measurement, but reaching the usual amount at others. Up to the maximum load no cracks or other signs of distress were visible. At the maximum load the shell or layer outside the spiral rapidly cracked in various places in the upper portion of the column shaft, the load fell about 10 per cent and the fireproofing shell dropped off from this portion, either at the maximum load or a little later with a continued movement of the testing machine. With further movement of the head of the testing machine, and a corresponding shortening of the column, the load again increased

and rose nearly to the original maximum load. During this time the spiral reinforcement in the exposed portion stretched as is usual with a hooped column, and the second maximum was that to be expected with a hooped column having a section corresponding to the outside diameter of the spiral.

It may be noted that the relation between the area of the gross section of a column having a given thickness of fireproofing, and the area of the section inclosed by the spiral will depend upon the size of the column. For these columns, the load carried after the fireproofing layer failed was nearly as much as that which caused the failure of the outer shell. For larger columns (with the same thickness of fireproofing and same percentage of reinforcement) the maximum load carried by a hooped column would be larger than the load which would produce failure of the fireproofing layer. For smaller columns the reverse would be true.

CONCLUSIONS.

The results of these tests warrant the following conclusions:

1. They seem to confirm the current practice as to the effect of varying percentages of longitudinal reinforcement within the range of these tests as expressed in the following formula:

$$f = f_c (1-p) + n f_c p$$

2. They indicate that columns with spiral reinforcement are stronger and tougher than columns with no spiral reinforcement.

3. For these tests it would appear that the spiral reinforcement is approximately four times as effective in giving ultimate strength to the column as the same volume of longitudinal reinforcement.

4. It appears that the effect of the longitudinal reinforcement decreases as the percentage of spiral reinforcement increases.

EXPLANATION OF PHOTOGRAPHS.

An excellent record of the appearance of the columns after testing is given in the photographs, Figs. 13 to 24 inclusive. It must be borne in mind, however, that in many of the columns containing spirals, the failure was carried to extremes by continuing the operation of the testing machine for several minutes after the ultimate load was reached. These exaggerated failures were produced to develop the planes of shear, because as a rule the spirally reinforced columns showed little damage under maximum loads.

The group photographs show the columns throughout their length. The enlarged ends which were a distinctive feature of these columns are clearly shown, as well as the portion of the columns in which failure developed.

Close views of the fractures are given in the individual photographs. The diagonal planes along which the columns sheared were about 50° to 55° in the spiral columns and about 65° in the columns having vertical rods only. The line of fracture across the spirals generally follows closely the shearing plane in the concrete, and where the spirals are broken the "necking-down" of the steel may be seen. In some cases, *e. g.*, Col. 5, there are evidences of "necking-down" without actual fracture.

Among other features of the detail views may be cited the soundness of the concrete except close to the fracture; the uniformity in spacing of the spirals; the buckling of the vertical rods, particularly in columns without spirals; the gage lines with their designating numbers and the steel plugs bedded in plaster of Paris for measurements in the concrete. Columns 3, 4 and 4*b* bulged uniformly in the locality of failure.

The plain columns in Group 7 indicate splintery fractures at the upper ends, but the usual diagonal planes were found upon removal of the tops. The lower halves of the plain columns were apparently unimpaired.

The damage done to the columns with only vertical rods is due to the maximum loads, as in all these cases the failures were sudden with a great reduction in the loads sustained. The failure of these columns was not exaggerated as in the case of the spirally reinforced columns. The buckling of the vertical steel may be seen to be localized between the horizontal ties.

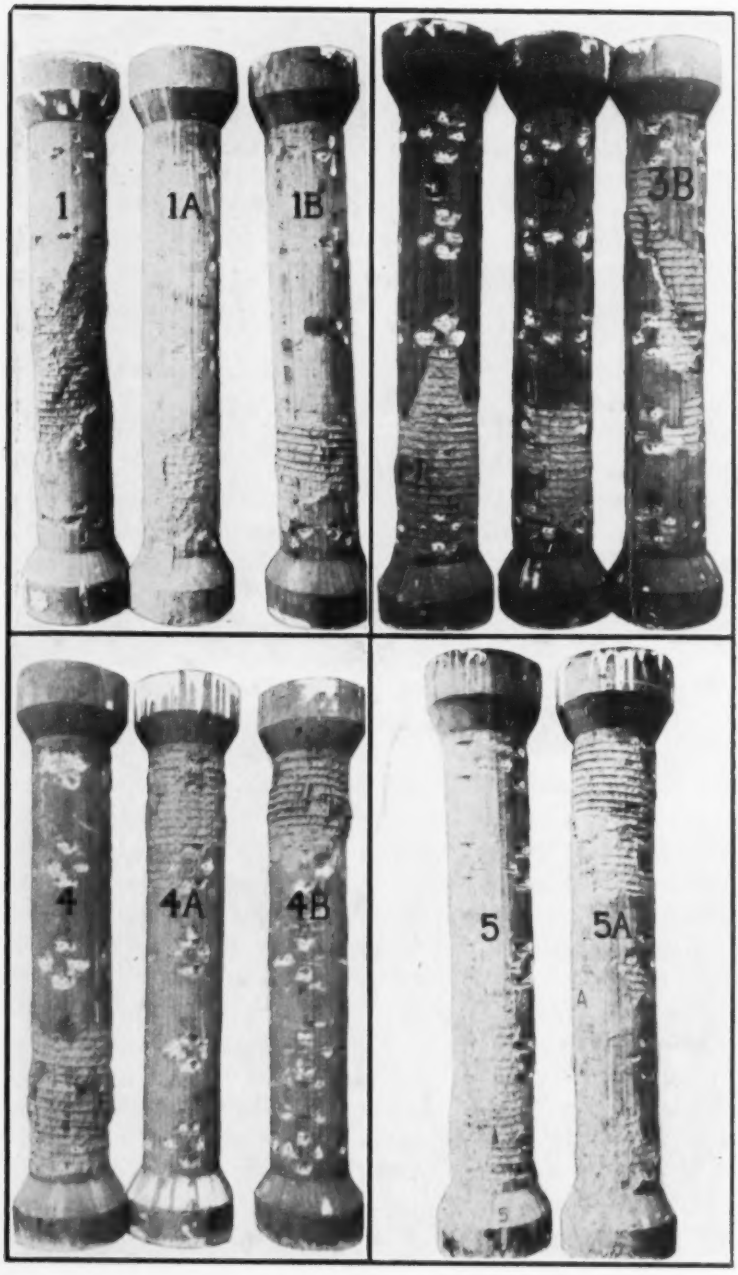


FIG. 14.—GENERAL VIEWS OF GROUPS 1, 3, 4 AND 5 AFTER TESTING.

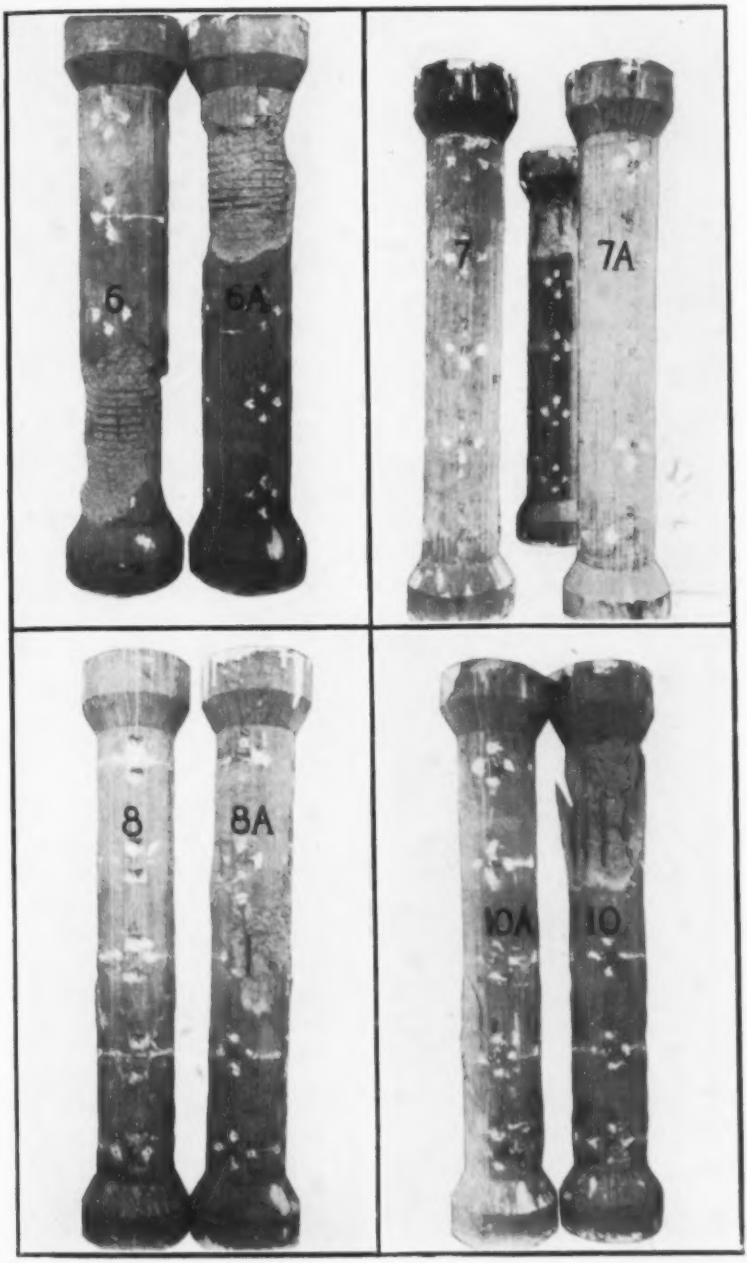


FIG. 15.—GENERAL VIEWS OF GROUPS 6, 7, 8 AND 10 AFTER TESTING.

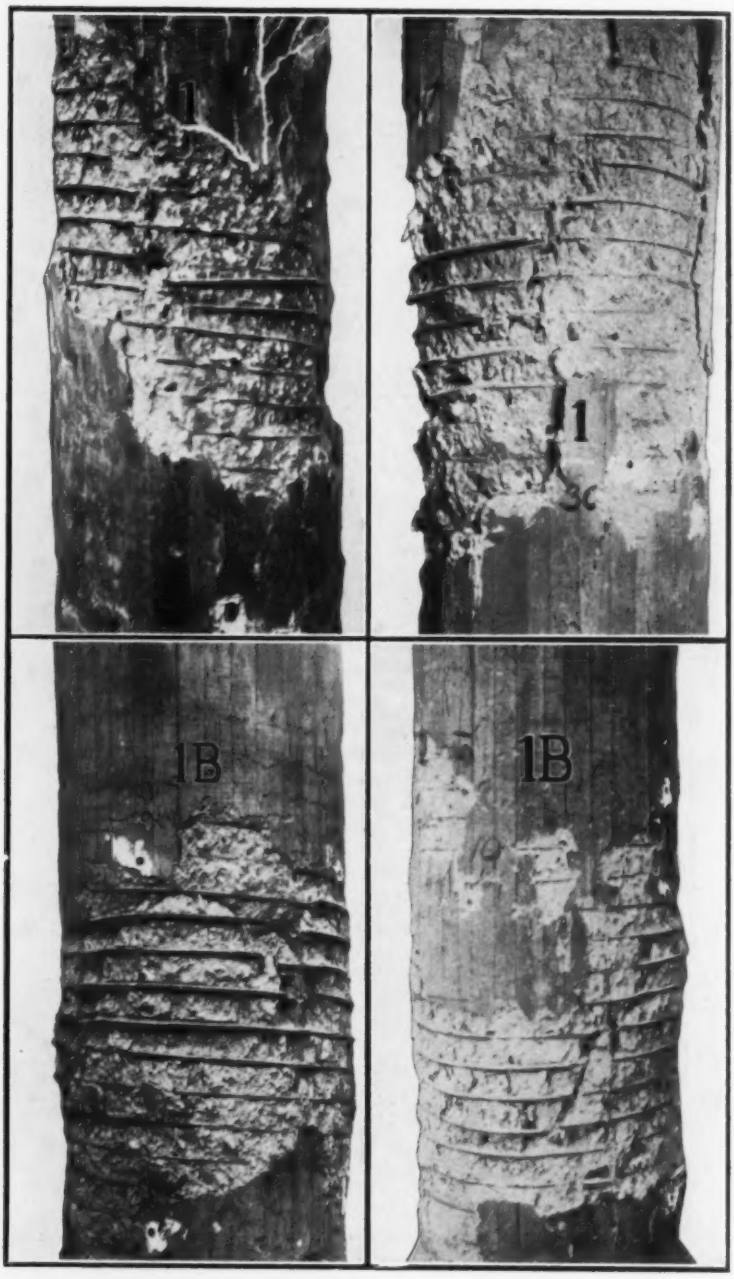


FIG. 16.—DETAIL VIEWS OF OPPOSITE SIDES OF COLUMNS 1 AND 1-B IN ZONE OF FAILURE.

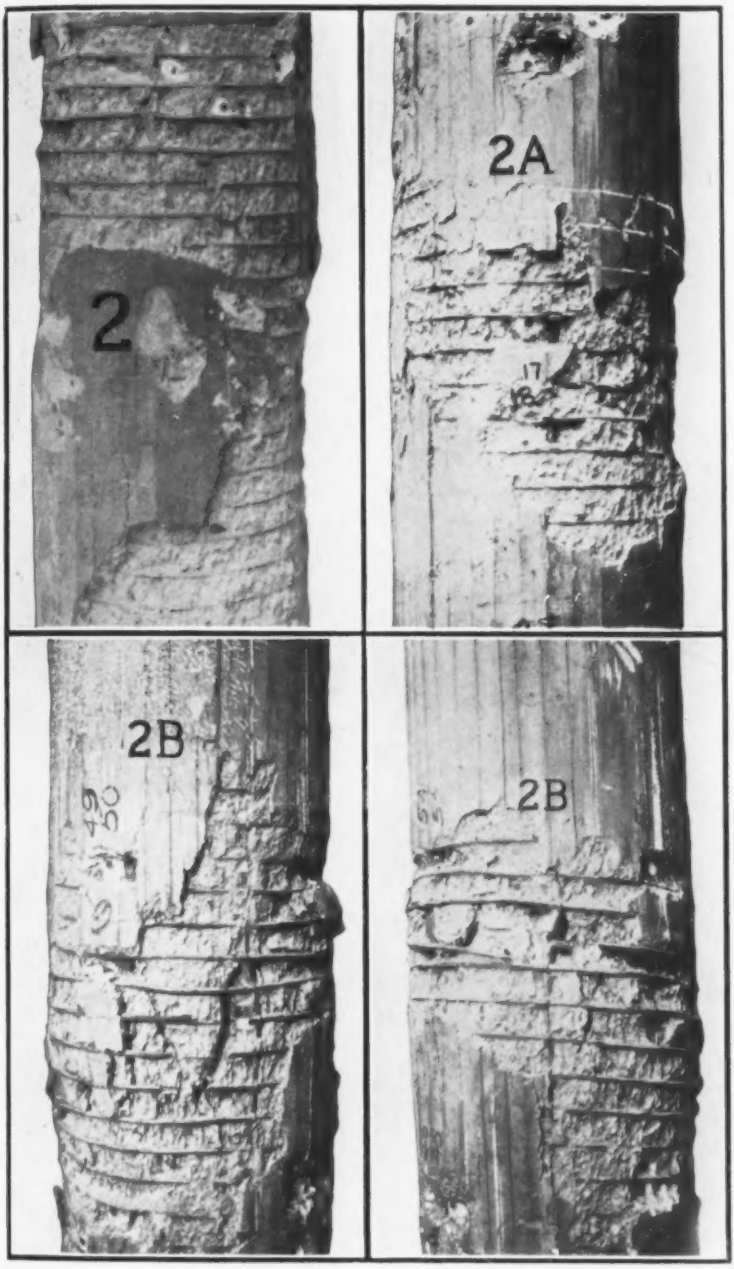


FIG. 17.—DETAIL VIEWS OF PORTIONS OF COLUMNS 2, 2-A AND OPPOSITE SIDES OF 2-B AFTER TESTING.

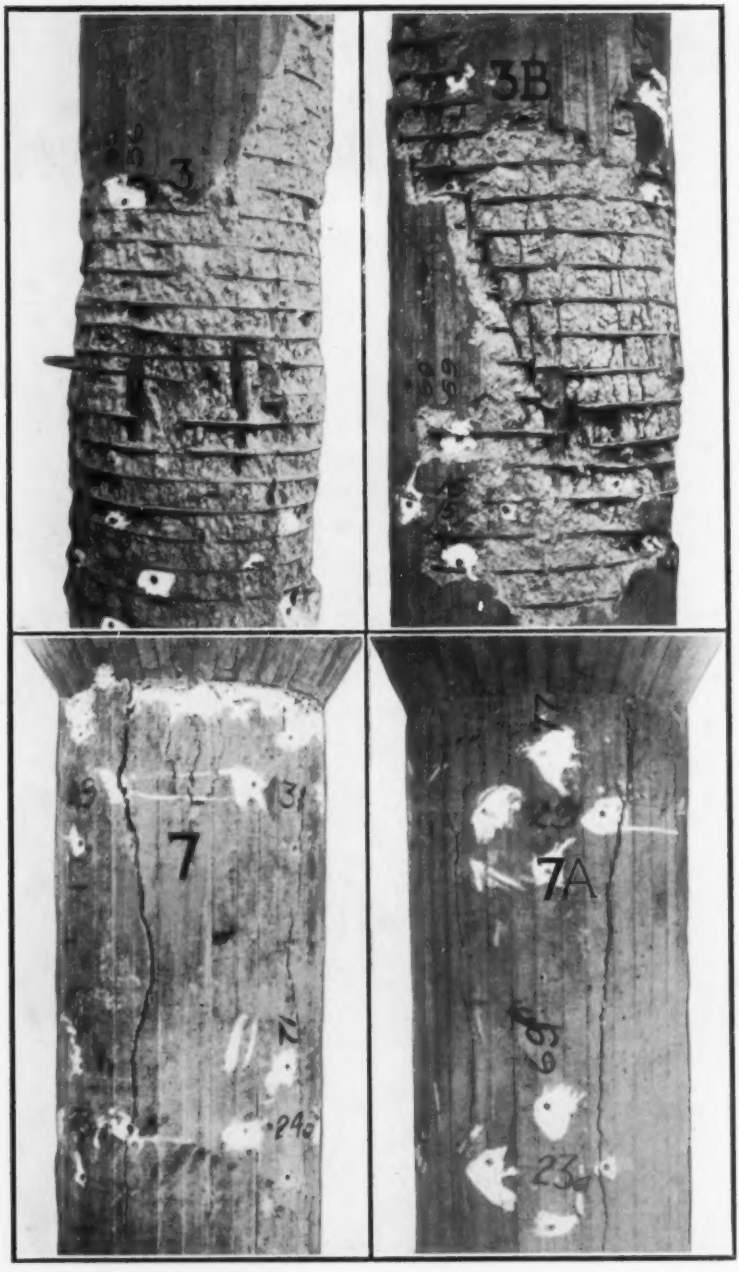


FIG. 18.—DETAILS OF FAILURE IN COLUMNS 3 AND 3-B WITH SPIRAL REINFORCEMENT, AND IN COLUMNS 7 AND 7-A WITHOUT REINFORCEMENT.

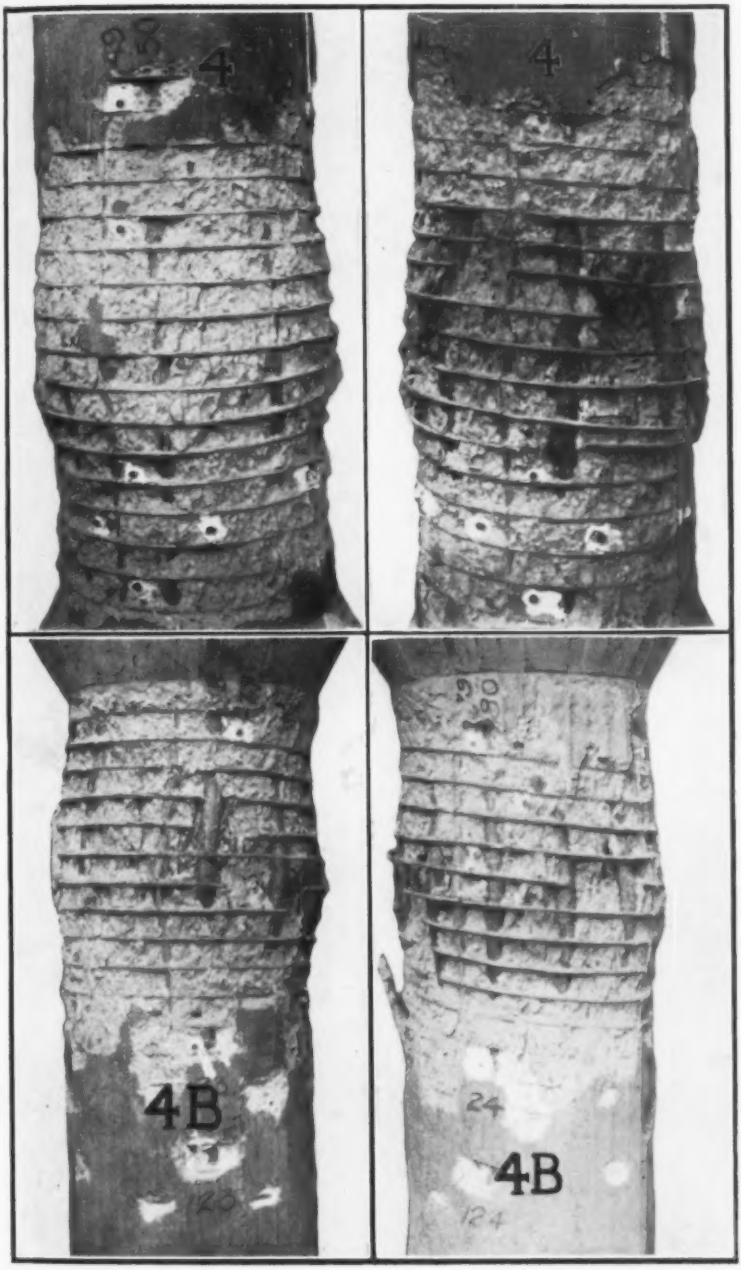


FIG. 19.—DETAIL VIEWS OF OPPOSITE SIDES OF COLUMNS 4 AND 4-B IN ZONE OF FAILURE.

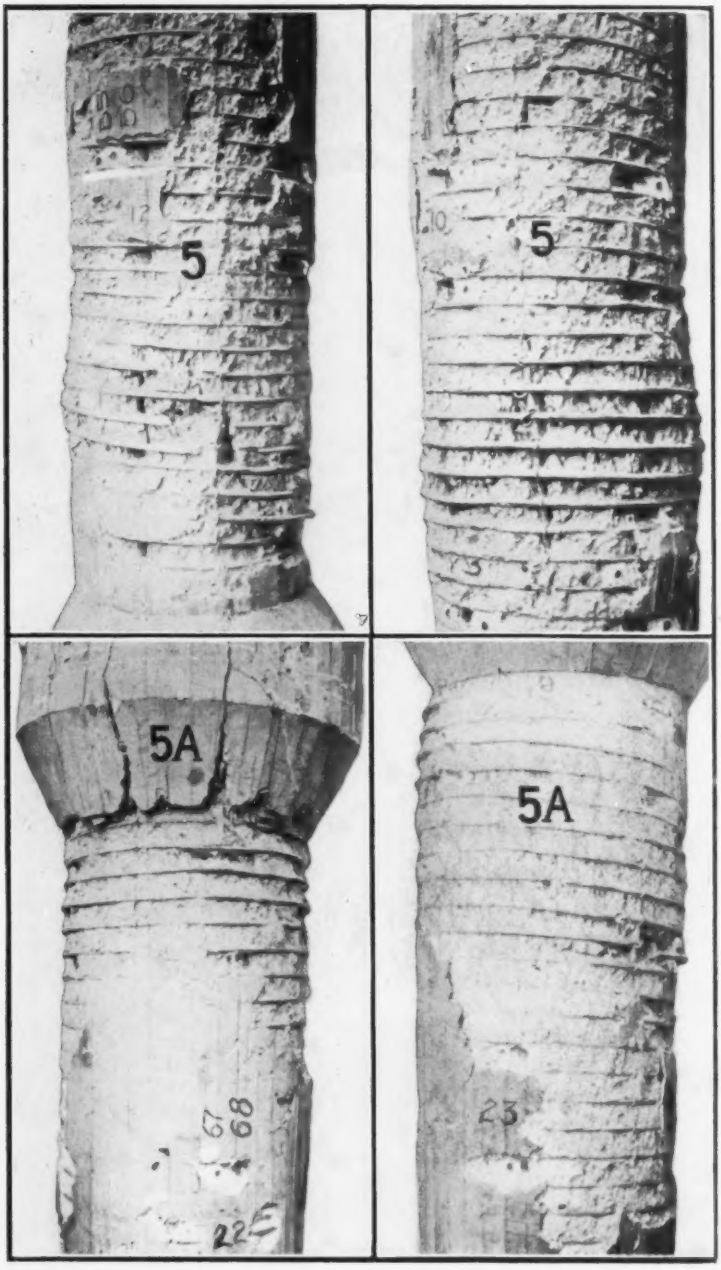


FIG. 20.—DETAIL VIEWS OF OPPOSITE SIDES OF COLUMNS 5 AND 5-A IN ZONE OF FAILURE.

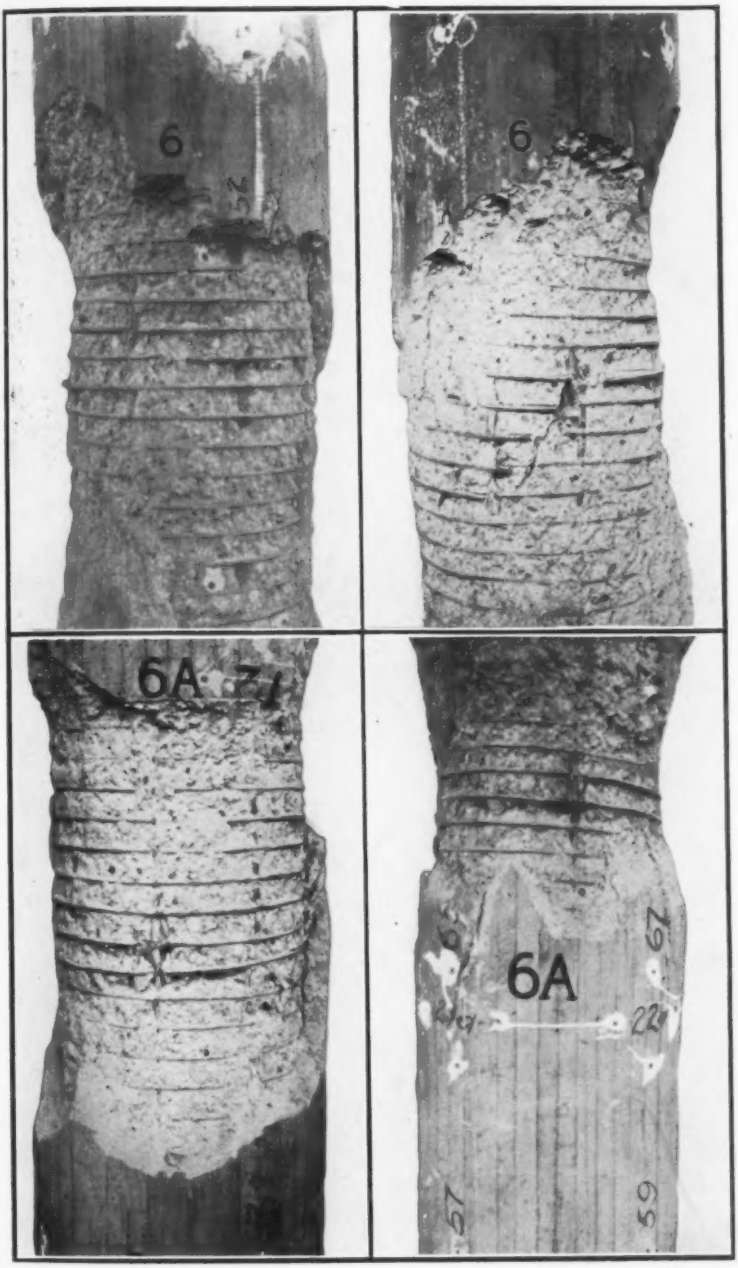


FIG. 21.—DETAIL VIEWS OF OPPOSITE SIDES OF COLUMNS 6 AND 6-A IN ZONE OF FAILURE.

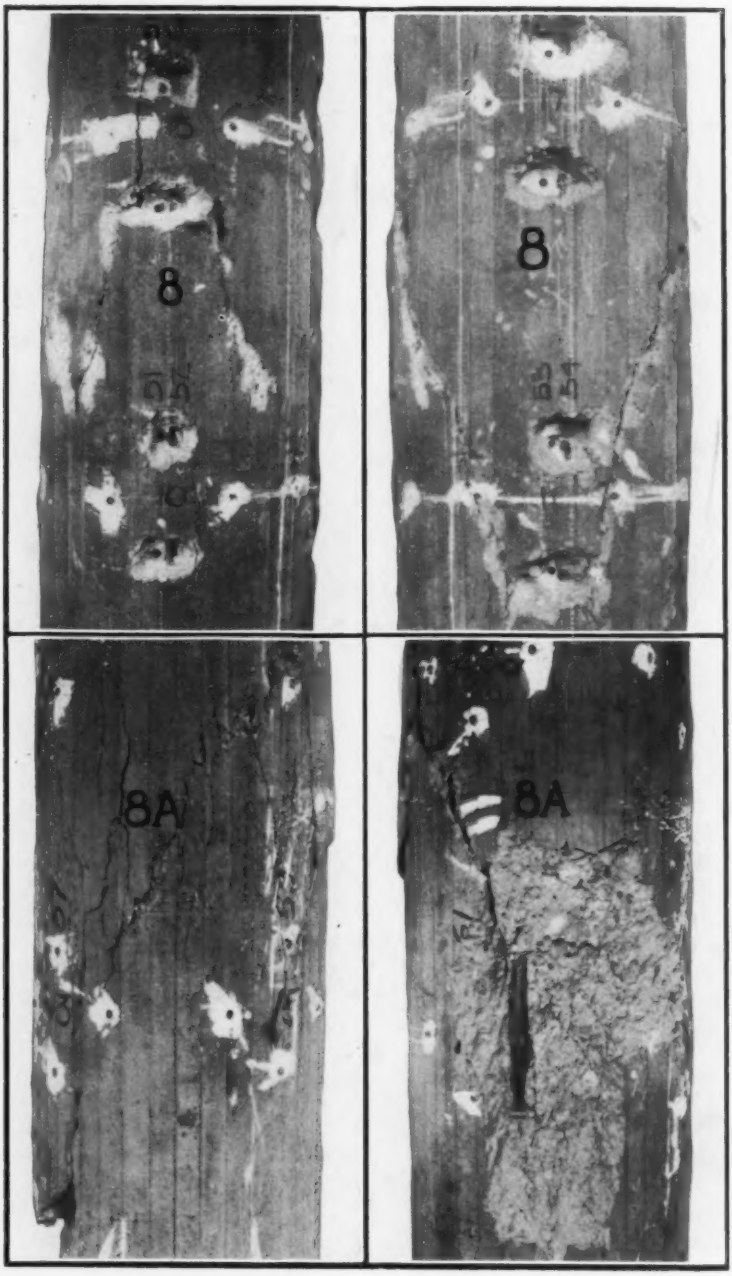


FIG. 22.—DETAIL VIEWS OF OPPOSITE SIDES OF COLUMNS 8 AND 8-A SHOWING CHARACTER OF FAILURE.

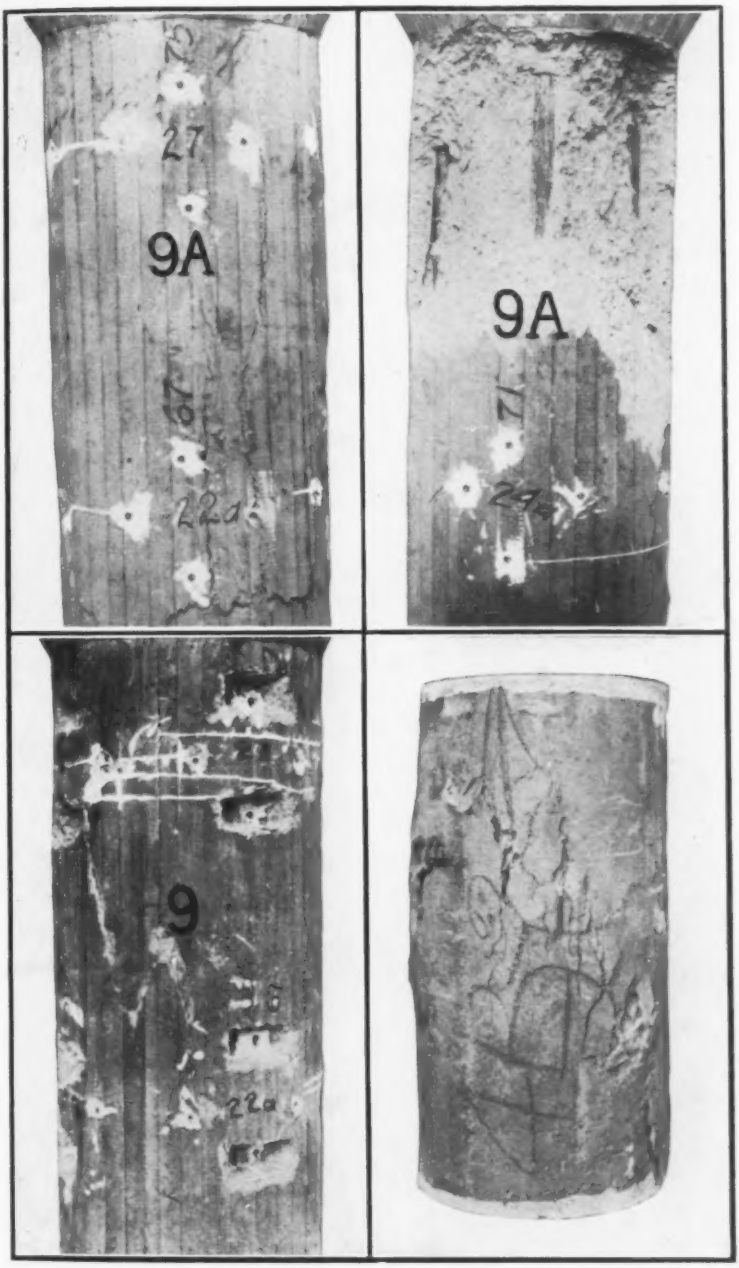


FIG. 23.—DETAIL VIEWS OF OPPOSITE SIDES OF COLUMNS 9 AND 9-A, AND OF A TYPICAL TEST CYLINDER AFTER FAILURE.

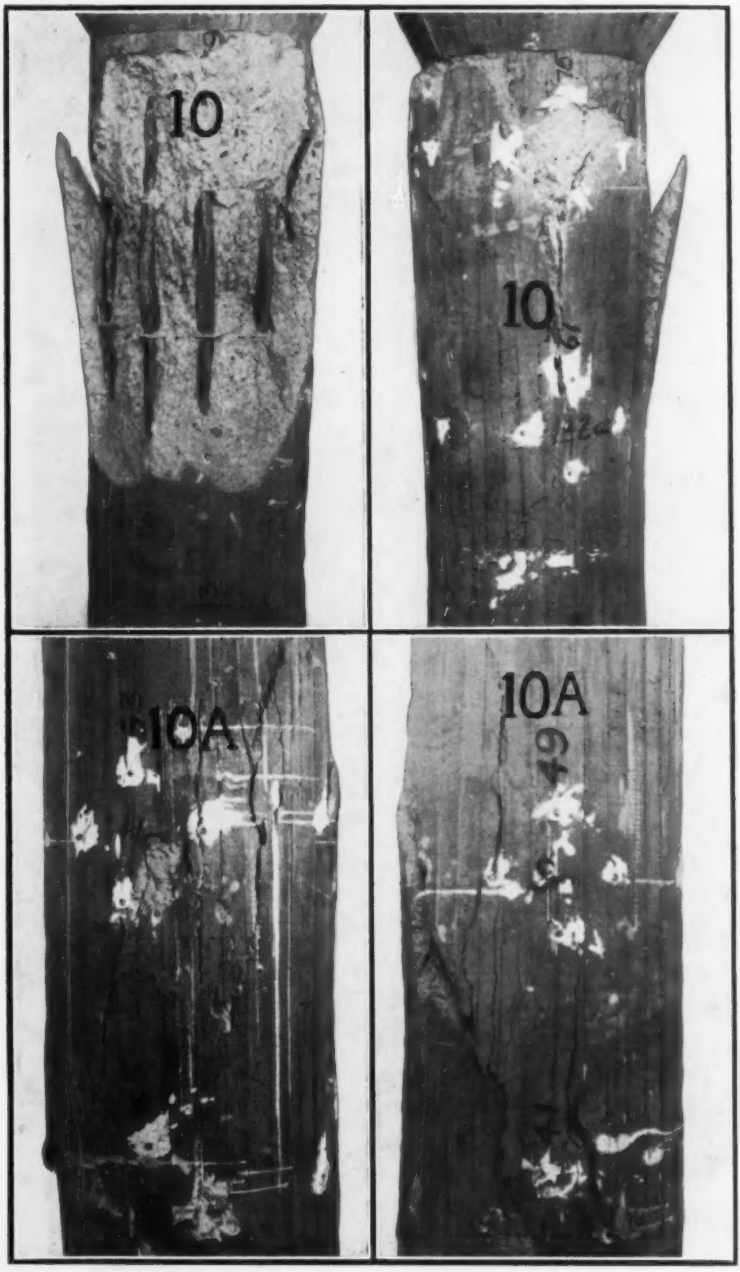


FIG. 24.—DETAIL VIEWS OF OPPOSITE SIDES OF COLUMNS 10 AND 10-A IN ZONE OF FAILURE.

EXPLANATION OF DIAGRAMS.

Figs. 25 to 48 inclusive show graphically the relation between total column loads and unit-deformations for all the columns as plotted from the data in Table XI. The several curves on each diagram are numbered and arranged in positions corresponding to the location of the gage lines on the columns as shown in Fig. 7.

Full lines represent gage-lines on the steel and dotted lines on the concrete.

Longitudinal stress-deformation curves are drawn to the left of the origin of co-ordinates, lateral stress-deformation curves to the right.

These diagrams do not show directly the actual stresses in the columns due to the loading, but these stresses may be readily determined. The stress in the spiral reinforcement corresponding to a given deformation as shown in the diagrams may be found by reference to Fig. 3, while the stress corresponding to a given deformation in the vertical reinforcement may be found from Fig. 4.

Stresses in the concrete in a longitudinal direction may be found only approximately by using the diagrams in Fig. 8. It is evident, however, that the lateral stresses in the concrete may not be determined from the deformations in Fig. 8 without a knowledge of Poisson's ratio.

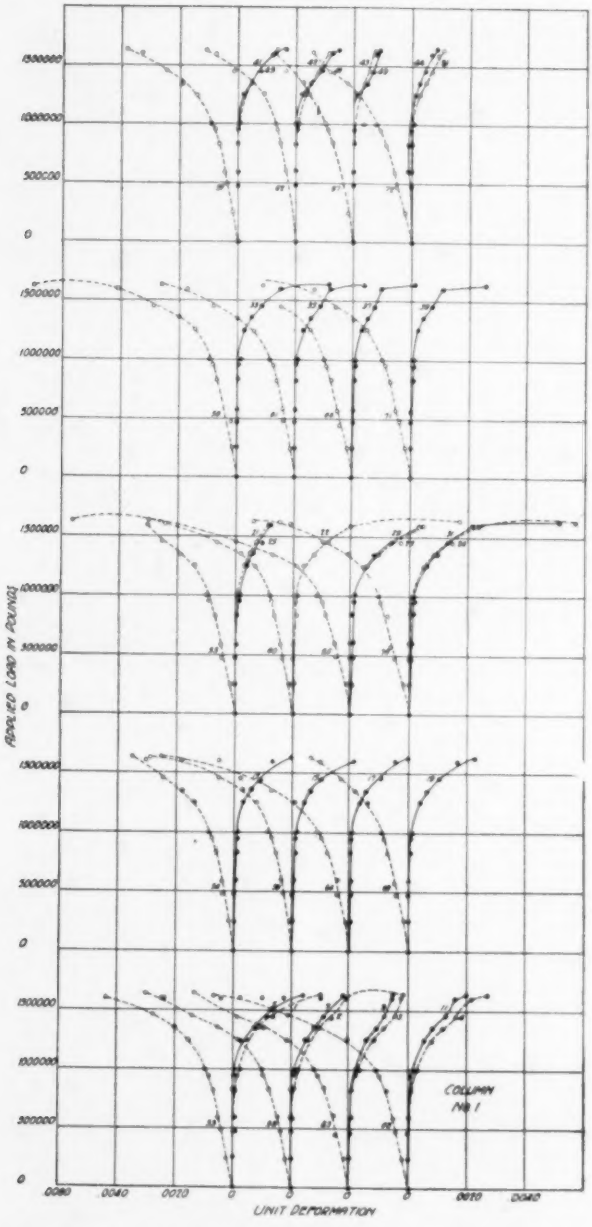


FIG. 25.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 1.

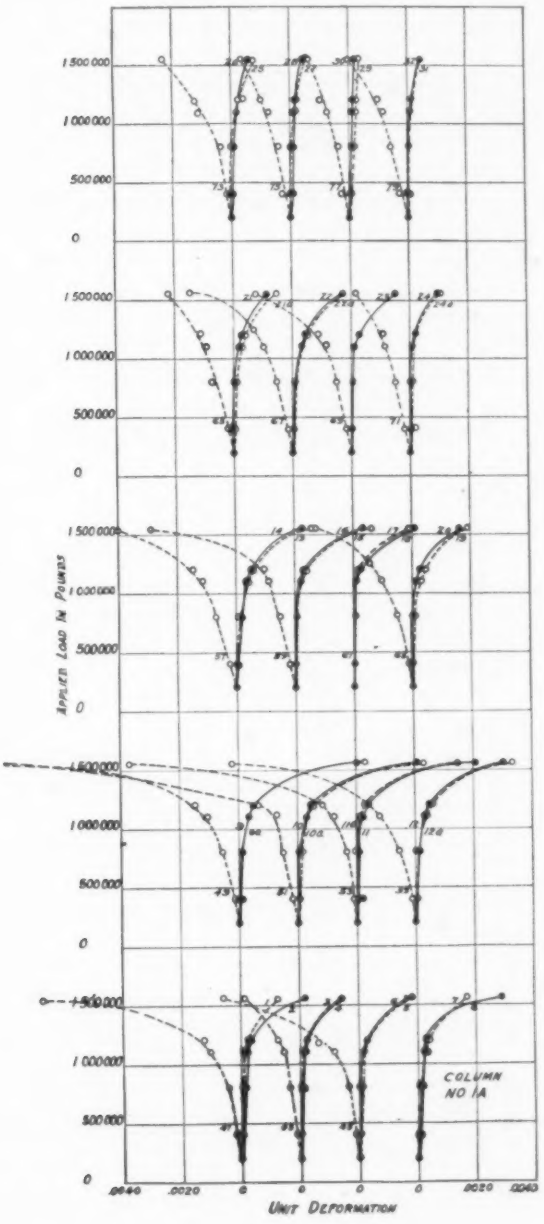


FIG. 26.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 1-A.

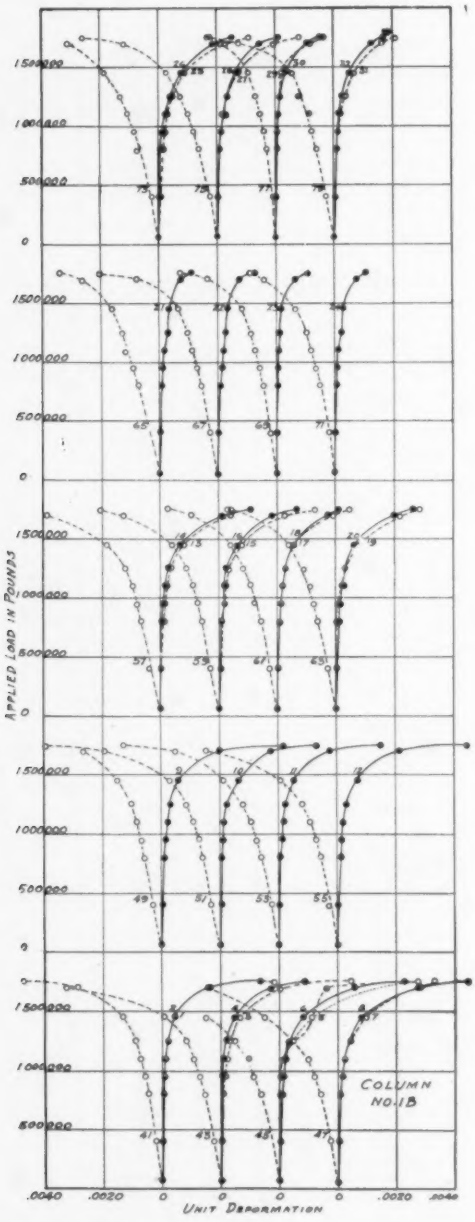


FIG. 27.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 1-B.

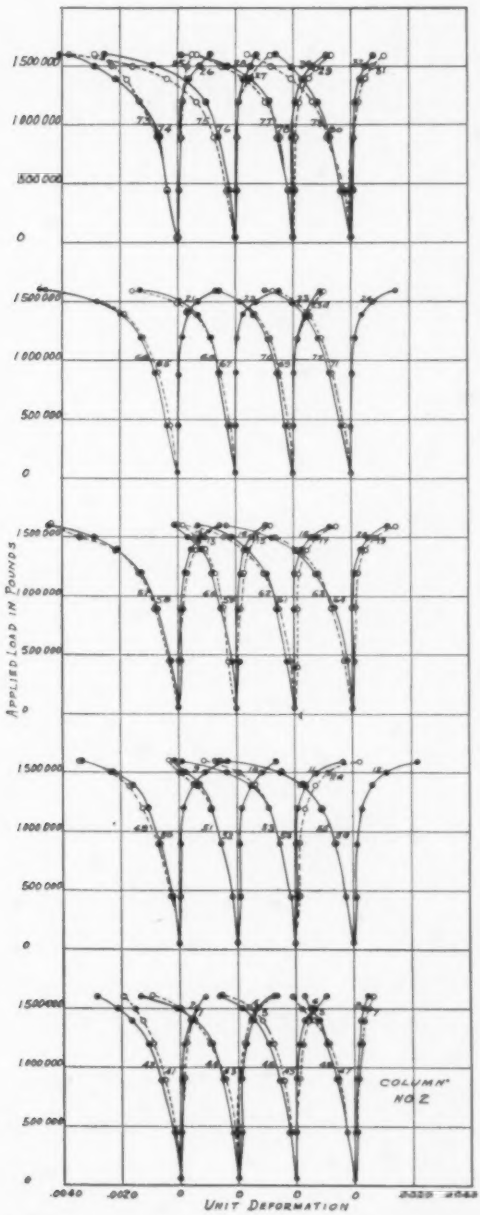


FIG. 28.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 2.

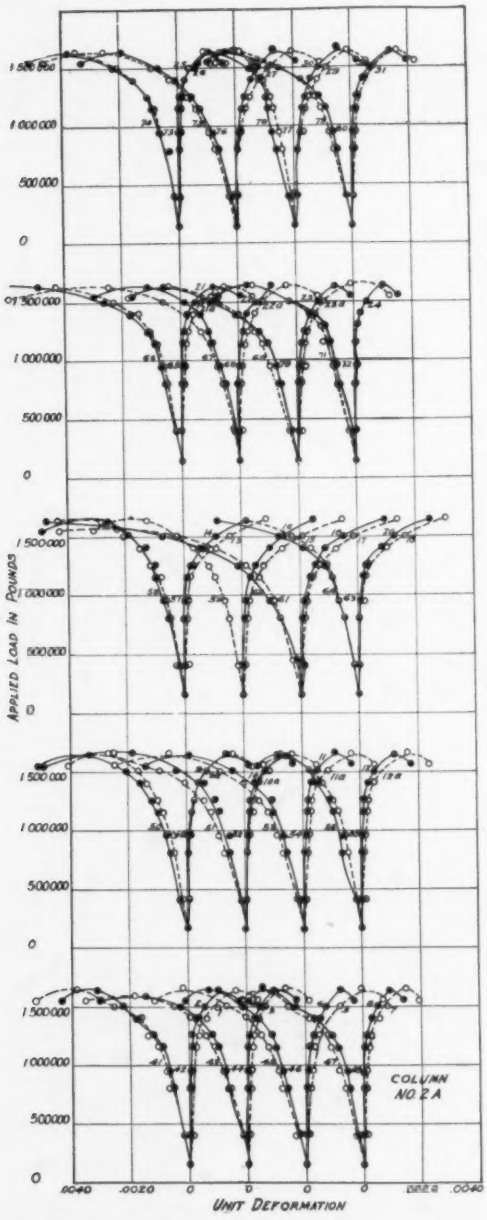


FIG. 29.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 2 A.

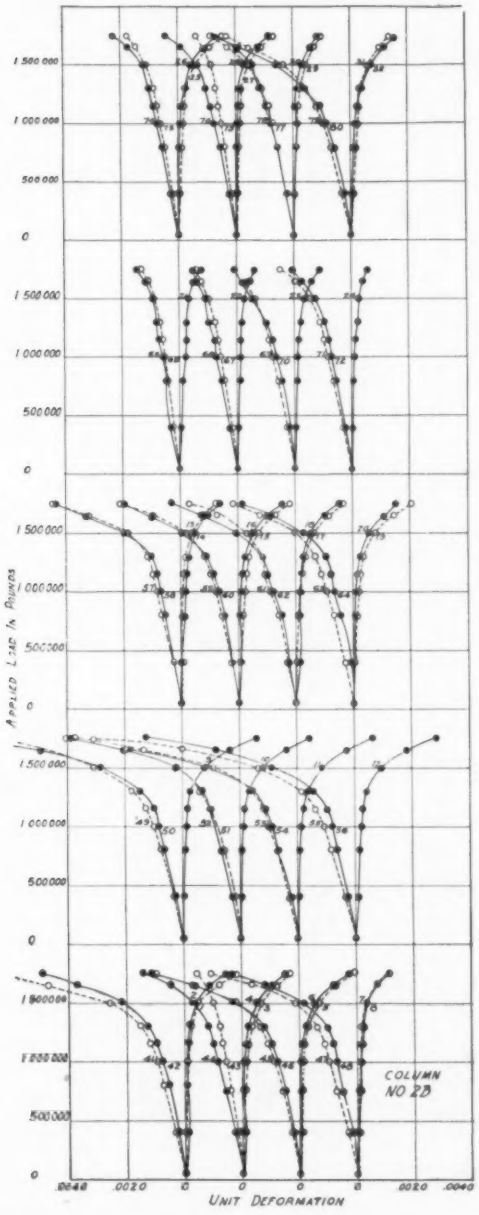


FIG 30.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 2-B.

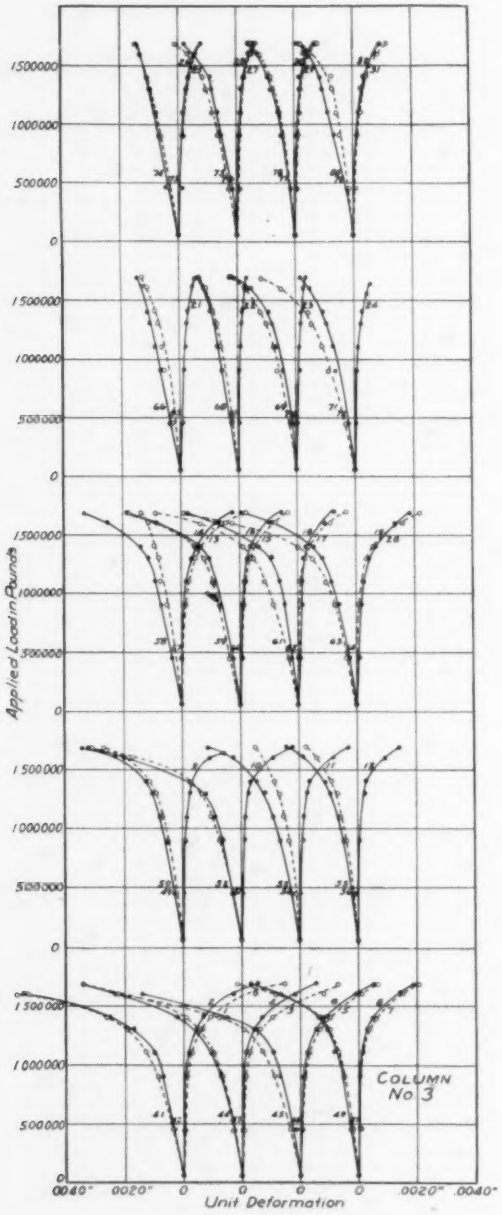


FIG. 31.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 3.

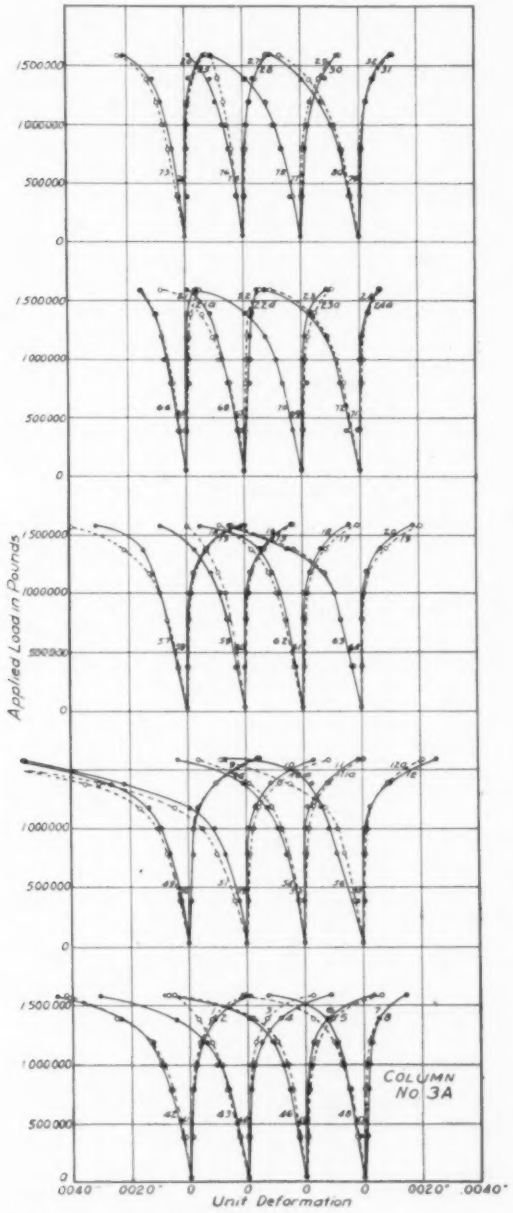


FIG. 32.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 3-A.

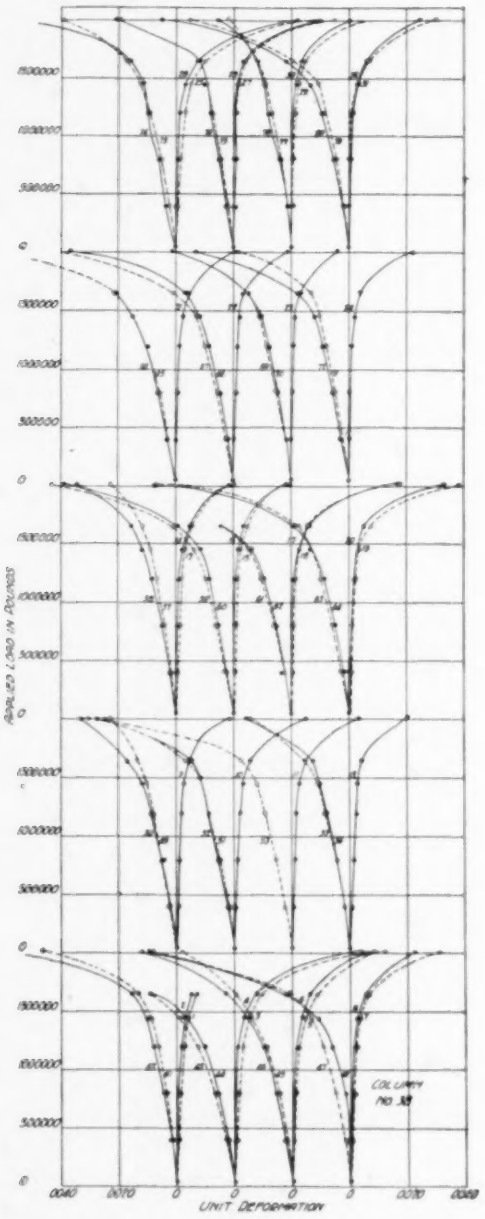
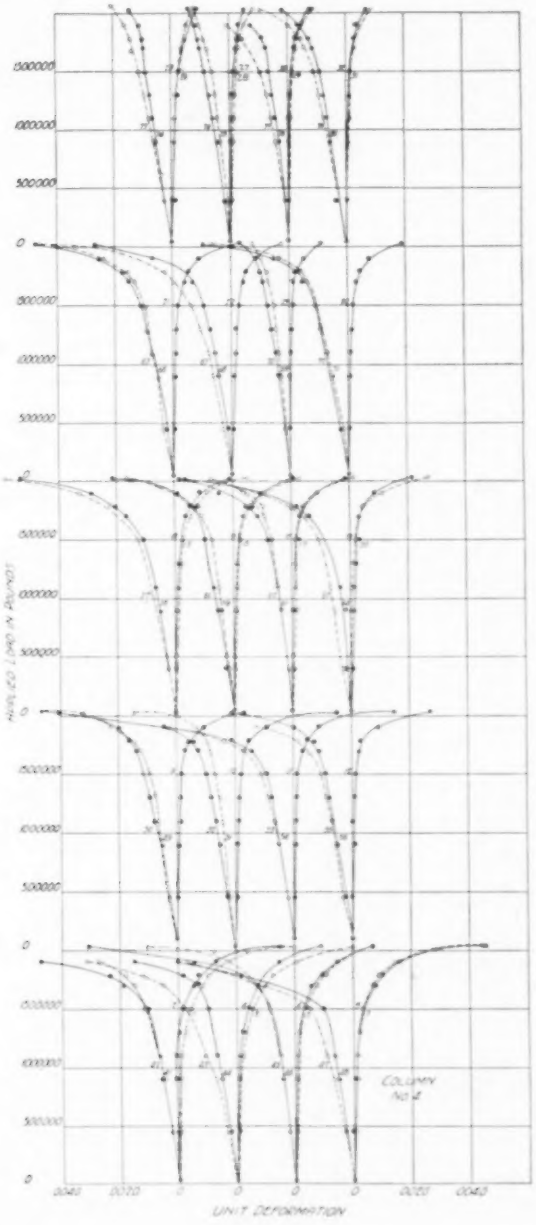


FIG. 33.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 3-B.


1500000
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500000
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19000000
19500000

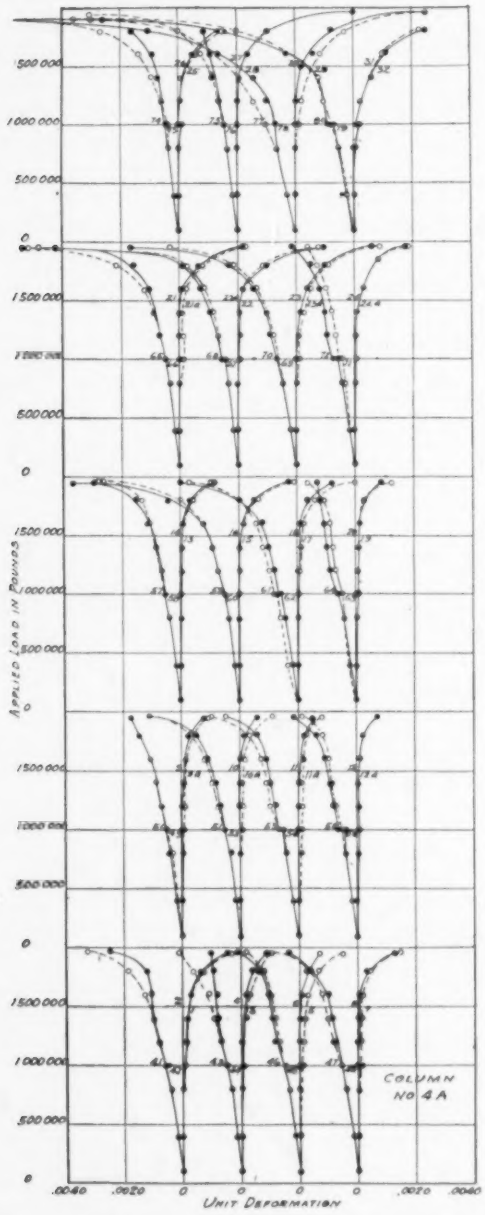

FIG. 35.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 4-A.

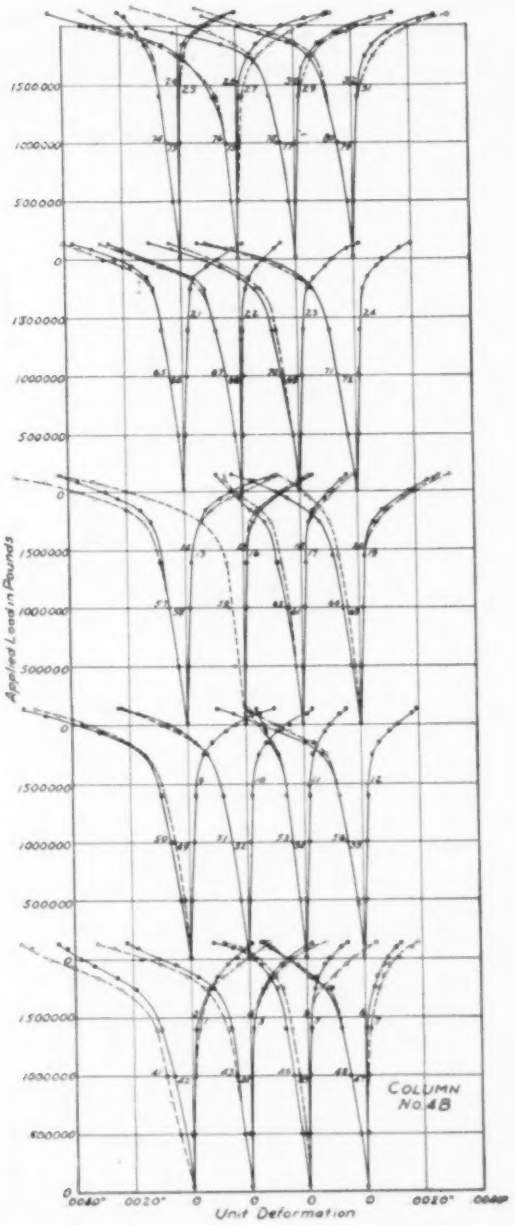

FIG. 36.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 4-B.

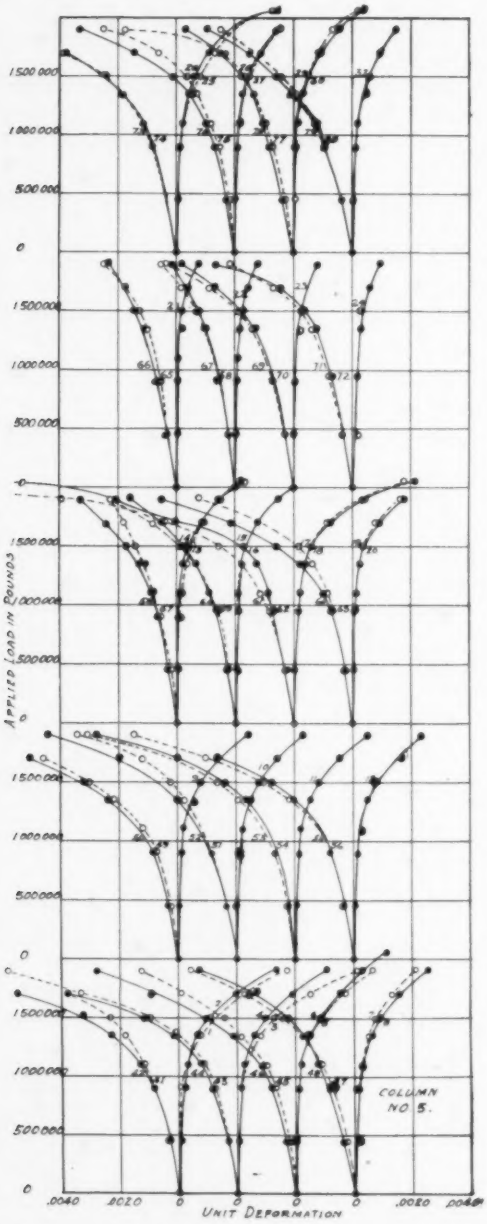

FIG. 37.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 5.

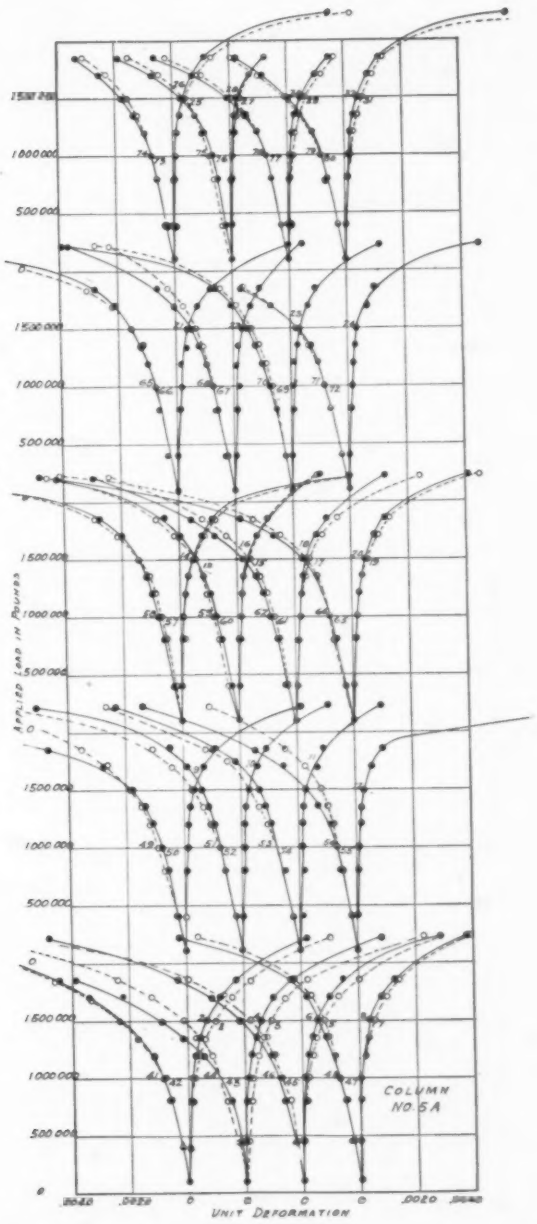

FIG. 38.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 5-A.

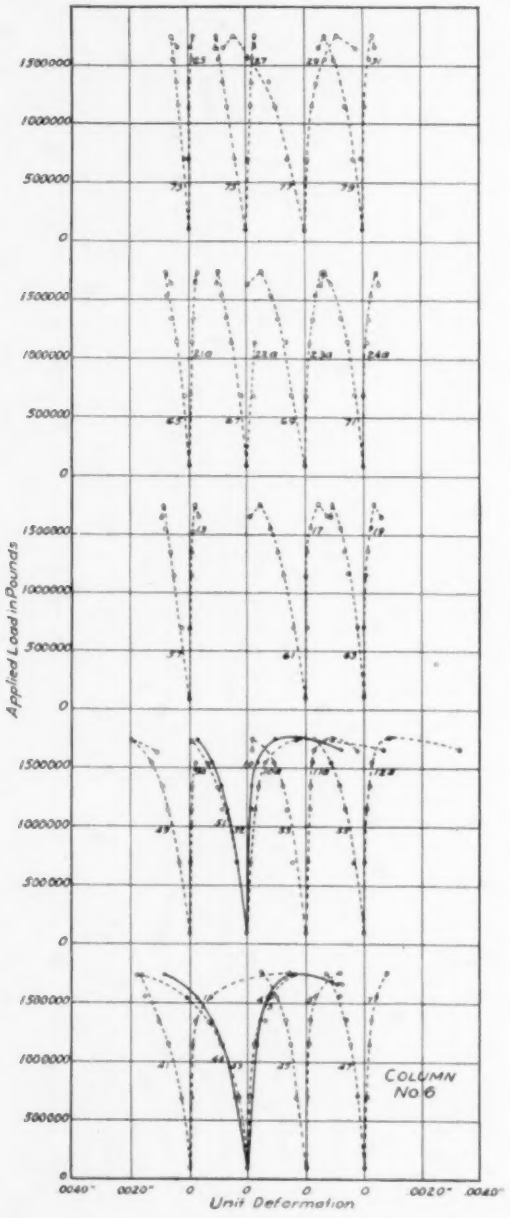

FIG. 39.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 6.

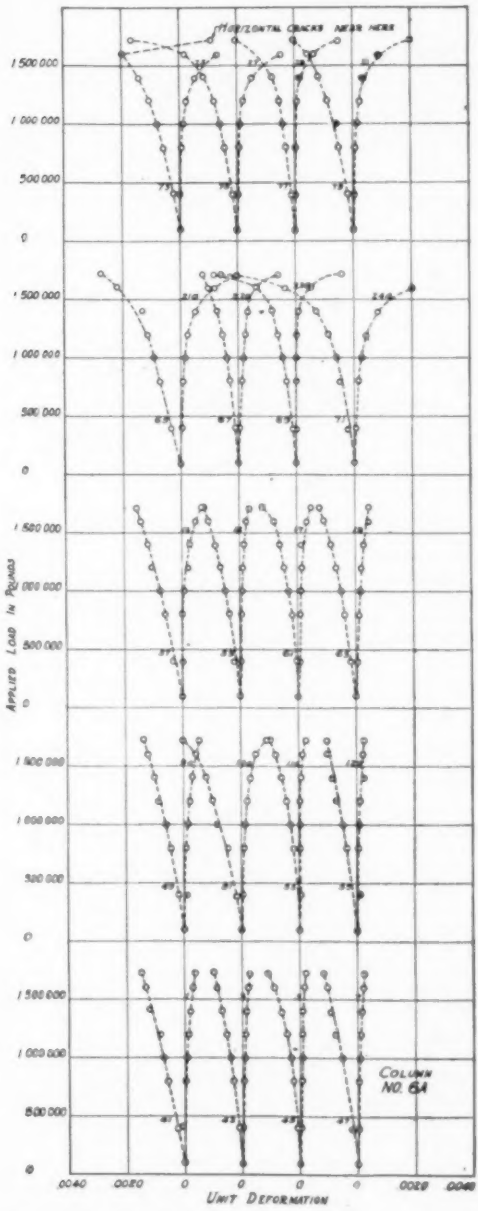

FIG. 40.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 6-A.

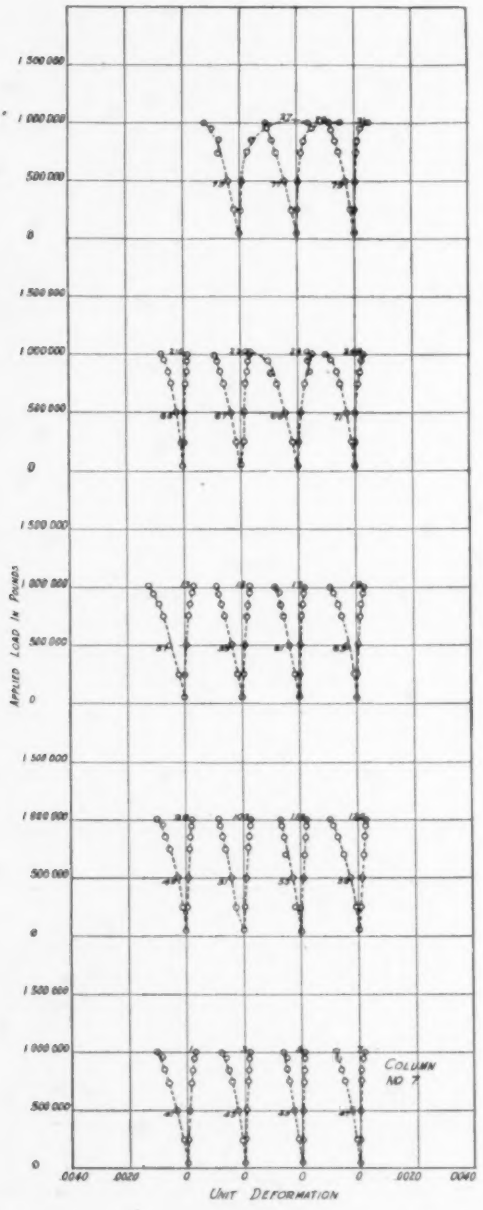

FIG. 41.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 7.

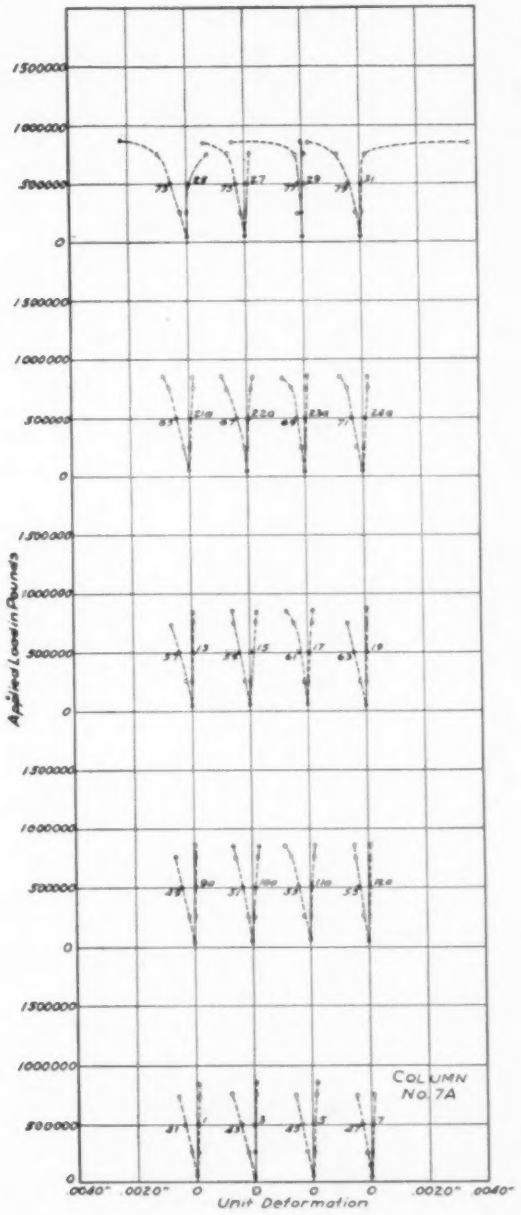

FIG. 42.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 7-A.

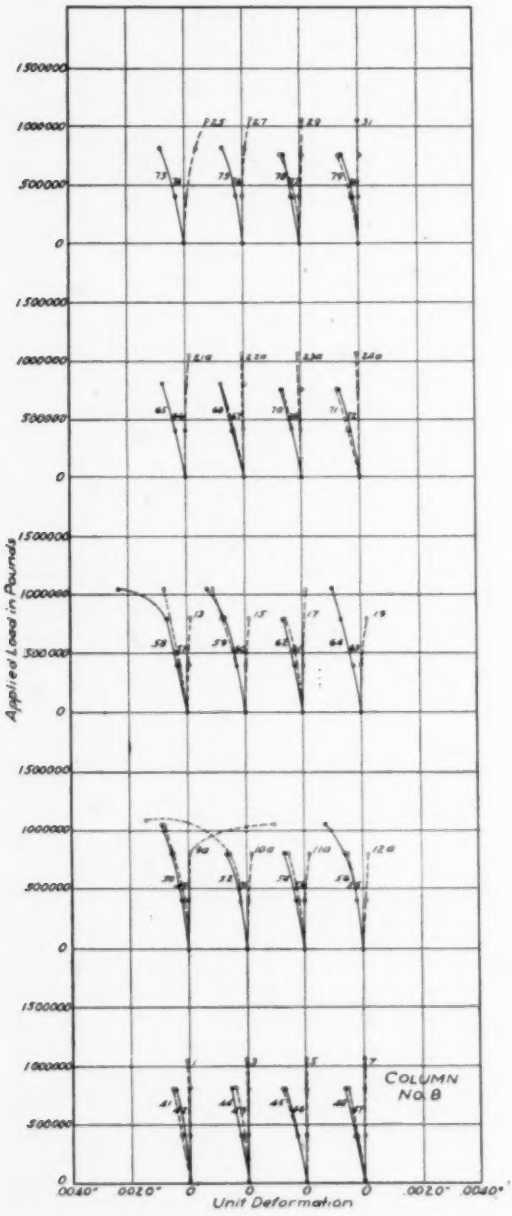

FIG. 43.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 8.

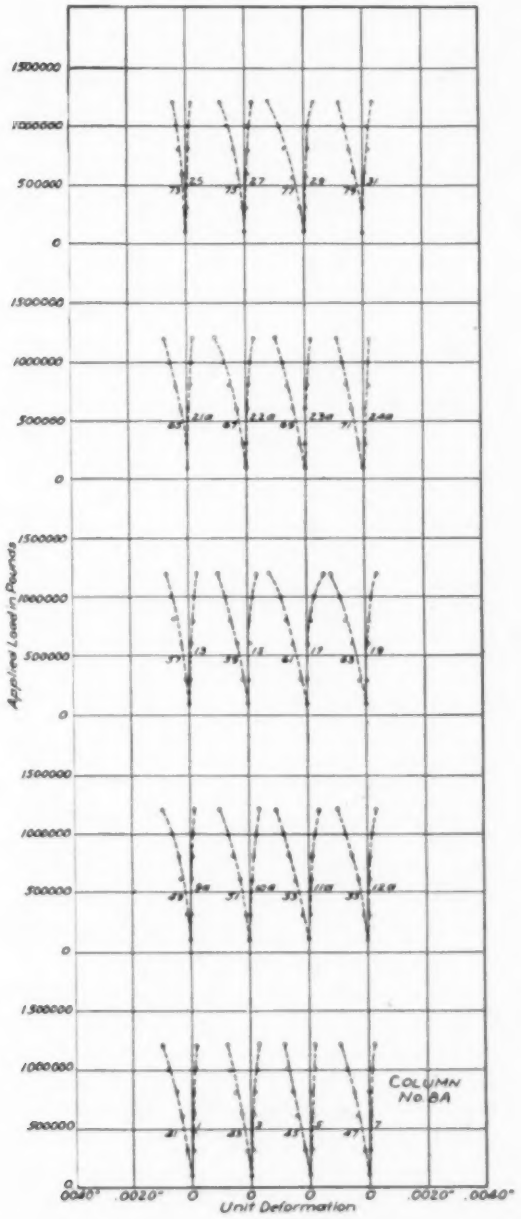

FIG. 44.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 8-A.

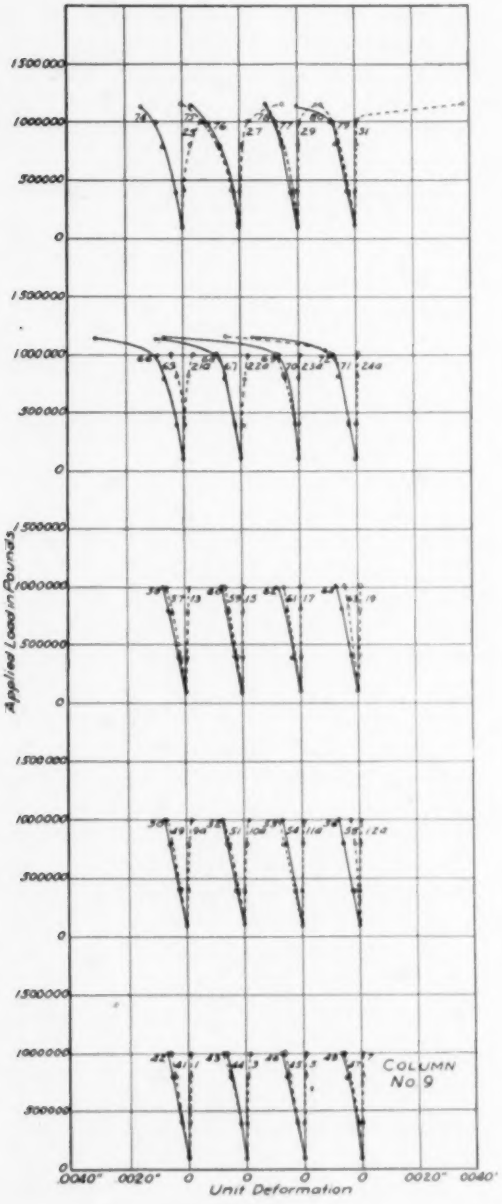

FIG. 45.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 9.

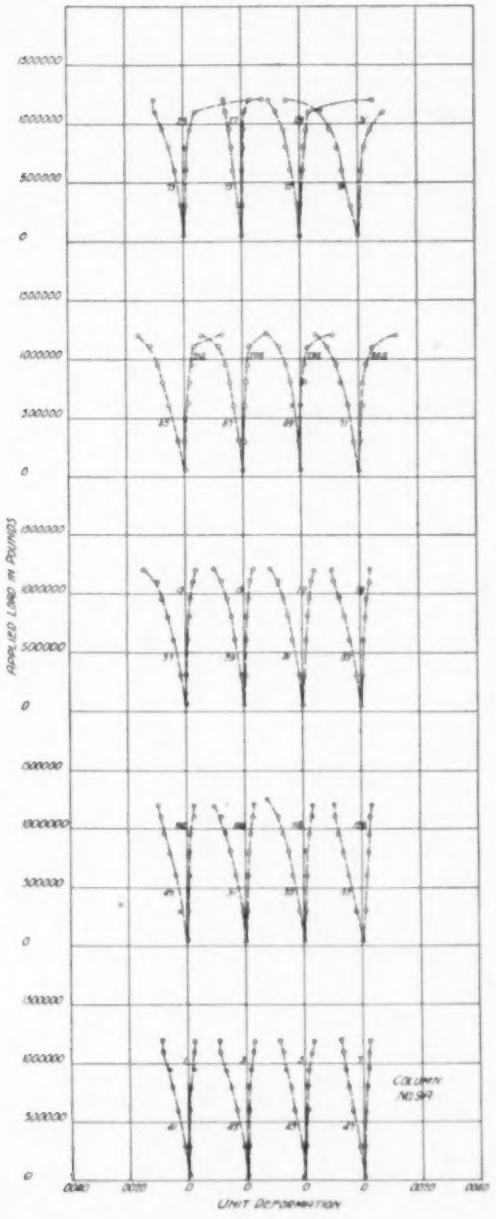

FIG. 46.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 9-A.

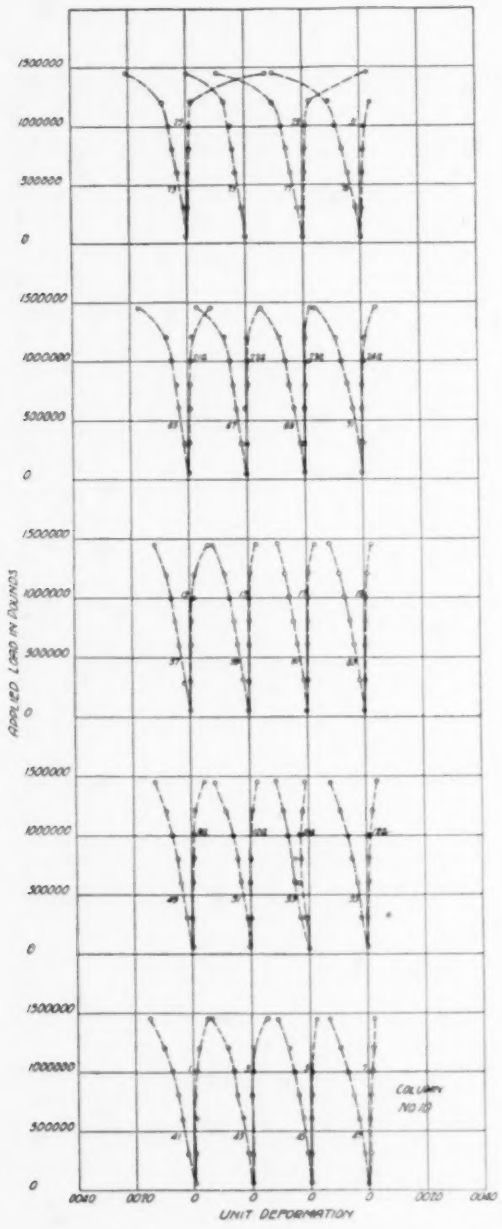

FIG. 47.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 10.

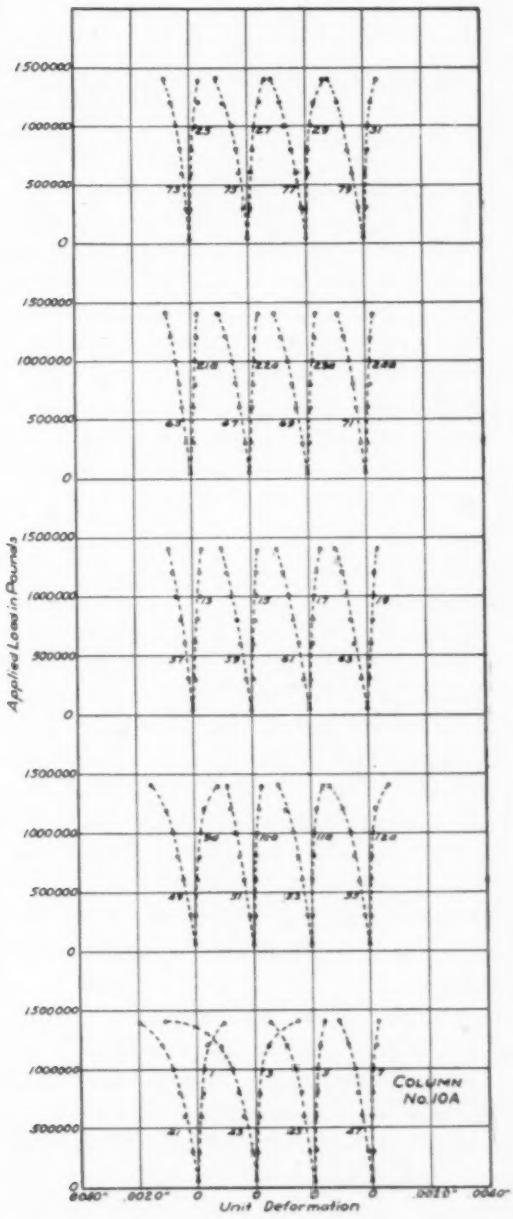

FIG. 48.—STRESS-DEFORMATION DIAGRAM, COLUMN NO. 10-A.

EXPLANATION OF TABLES.

The following tables present—

First, the initial loads to which the test columns were subjected when strain-gage readings were first made; and second, the unit-deformations on all the lateral and longitudinal gage lines, between the initial load and the successive applied loads as given in the tables.

The gage lines were numbered as shown in Fig. 7, with the exception of column No. 1. The letters "C" and "S" were added to the gage line numbers to indicate at a glance whether the line was in the concrete or on the steel. Supplementary gage lines not provided for in the general scheme of numbering include the letter "a" in the gage line number; thus, G. L. 22a C was in the concrete adjacent to G. L. 22 S.

Gage lines were established for Col. 1 in a similar manner to the general scheme in Fig. 7, except that the gage lines on the spirals were laid off consecutively end to end for one full turn of the spiral. The closures between the initial and final points on the spirals were slightly out of the range of the strain gage and therefore the readings on such gage lines were omitted. The numbering for Col. 1 starts at the bottom and progresses around the column from left to right, first on the concrete and then on the spiral. The vertical gage line numbers start from the same quarter and follow in sequence from bottom to top on the east, north, west and south faces respectively.

The loads recorded in the tables were those at which the testing machine was in balance at the instant of stopping to take strain-gage readings.

Series 1 and 2 (also series 3 for columns 1 and 10a) were the zero readings on the gage lines as taken under the initial load.

The unit-deformations are stated in hundred-thousandths of an inch. These have been computed from the observed deformation in a gage length of 8 in. by dividing the same by the gage length and the multiplication ratio of the strain-gage and correcting it for temperature. Positive values indicate elongations and negative values contractions.

No gage lines were established on the vertical steel on columns 6, 6A, 8A, 9A, 10 and 10A, inasmuch as the large amount of cutting required would probably have affected the carrying capacity of the columns seriously. Such gage lines were established in columns 8 and 9, reducing the cross-section by approximately 40 sq. in.

Inspection of the tables shows the important fact that even at ultimate loads the amount of shortening in the spirally reinforced columns is insignificant from a practical standpoint.

TABLE XI.—SHOWING INITIAL LOADS IN POUNDS.

Col. No.	Initial Load.	Col. No.	Initial Load.	Col. No.	Initial Load.
1	10 000	3B	70 000	7	50 000
1A	205 000	4	50 000	7A	50 000
1B	70 000	4A	100 000	8	58 000
2	90 000	4B	50 000	8A	100 000
2A	140 000	5	0	9	100 000
2B	50 000	5A	100 000	9A	50 000
3	45 000	6	110 000	10	50 000
3A	30 000	6A	90 000	10A	50 000

Deformations due to the application of above loads should be calculated by extrapolation and added to the deformations observed above the initial load to obtain total deformation at any load.

Column 3B was subjected to repetitive loading in order to observe the amount of set produced by different loads.

Column 4B had eight additional series of gage lines, Nos. 101S to 132S inclusive, located on spirals intermediate between the zones of measurement in the standard lay-out.

The unit-deformations show very few apparent irregularities and these may be due in part to accidental errors in observing or recording. Every computation has been checked independently and the tables are in every respect as reliable as the original strain-gage readings besides being in much more convenient form for study.

COLUMN NO. 1.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		1 C †	2 C	3 C	4 C	5 S	6 S	7 S	8 S	9 S	10 S	11 S	12 S	13 S	13a C	14 S
242	4	+003	+003	+002	+003	+003	+001	+001	+002	000	+002	+003	Gage	+003	+001	+002
479	5	+004	+006	+006	+004	+001	+004	+003	000	+003	+003	+003	line	+006	+004	+002
585	6	+006	+007	+010	+006	+003	−008	+001	+004	+007	+006	+008	not	+008	+011	+001
822	7	+009	+013	+012	+012	+004	+008	+005	+011	+006	+007	+008	laid	+009	+014	+011
945	8	+014	+017	+025	+020	+007	+012	+015	+013	+018	+015	+016	off	+012	+014	+013
990	9	+016	+022	+030	+027	+013	+014	+017	+021	+024	+018	+022		+015	+011	+010
1250	10	+053	+057	+078	+075	+027	+038	+050	+055	+061	+061	+058		+033	+034	+033
1361	11	+092	+095	+118	+119	+077	+063	+078	+086	+096	+092	+087		+059	+058	+060
1440	12	+148	+139	+155	+165	+118	+096	+111	+103	+120	+118	+127		+093	+094	+085
1440	13*															
1616	14	+103	+188	+179	+223	+177	+295	+100	+123	+146	+112	+163		+135	+134	+136
1621.5	15	+300	+383	+182	+278	+240	+192	+178	+128	+144	+163	+204		+199	+188	+216

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		15 S	17 S	18 S	19 S	20 S	21 C	22 C	23 C	24 C	25 S	26 S	27 S	28 S	29 S	30 S
242	4	+008	+001	+003	+008	+005	+002	+007	+005	+002	+003	−003	Gage	Gage	+007	+003
479	5	+006	+002	+004	+004	+005	+002	+005	+001	+003	+007	−001	line	line	+006	000
585	6	+007	+003	+007	+011	+005	+007	+008	+001	+003	+005	+003	not	not	+010	+003
822	7	+015	+003	+014	+010	+008	+005	+012	+008	+008	+009	+002	laid	laid	+011	+005
945	8	+011	+007	+015	+012	+008	+008	+013	+012	+012	+015	+010	off	off	+015	+010
990	9	+015	+008	+017	+016	+013	+009	+016	+018	+016	+018	+009			+018	+015
1250	10	+040	+035	+037	+043	+030	+029	+037	+055	+048	+034	+023			+049	+046
1361	11	+064	+058	+067	+067	+056	+055	+071	+093	+087	+063	+047			+083	+081
1440	12	+100	+105	+107	+109	+093	+089	+118	+155	+157	+085	+081			+103	+136
1440	13*						+069	+123	+158	+151	+093	+078			+136	
1616	14	+164	+155	+164	+168	+123	+108	+205	+248	+244	+119	+121			+238	+245
1621.5	15	+216	+199	+210	+233	+184	+160	+574		+571	+163	+235				

* Series 13 was taken after load remained on column over night, i. e., 16½ hours.

† Throughout the tables these figures indicate gage line numbers.

COLUMN NO. 1—(*Continued*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.													
		31 S	32 S	33 S	34 S	35 S	36 S	37 S	38 S	39 S	40 S	41 C	42 C	43 C	44 C
242	4	+003	−002	+003	000	+003	Gage	+003	Gage	−007	+001	+001	+003	+001	000
479	5	+007	+003	+003	+008	+004	line	+005	line	−306	+004	−003	+004	−003	−001
585	6	+009	+008	+005	+005	+004	not	+006	not	−003	+009	−004	−003	−002	+002
822	7	+013	+008	+005	−003	+007	laid	+007	laid	+013	+013	−001	+005	−001	+006
945	8	+018	+008	+012	+008	+008	off	+011	off	+010	+009	+006	+008	+003	+008
990	9	+018	+018	+014	+010	+007		+012		+012	+011	+008	+013	+041(†)	+017
1250	10	+058	+061	+025	+021	+030		+028		+026	+030	+029	+028	+024	+033
1361	11	+093	+074	+050	+041	+052		+050		+044	+050	+049	+051	+038	+052
1440	12	+145	+114	+084	+081	+093		+076		+077	+084	+081	+083	+059	+070
1440	13*	+134													
1616	14	+223	+145	+149	+104	+122		+097		+107	+128	+124	+113	+071	+096
1621.5	15	+520	+191	+314	+287	+243		+214		+261	+305	+173	+141	+074	+111

Total Load, in Thousands of Pounds.	Series.	Lateral.								Longitudinal.					
		45 S	46 S	47 S	48 S	49 S	50 S	51 S	52 S	53 C	54 C	55 C	56 C	57 C	58 C
242	4	−003	−003	−007	Gage	000	−010	−008	−003	−018	−013	−009	−010	−016	−017
479	5	−004	−003	−009	line	+002	−007	−010	−006	−040	−039	−040	−026	−036	−042
585	6	−004	−003	−009	not	+008	−008	−010	−002	−048	−044	−047	−048	−045	−051
822	7	−003	−001	−006	laid	+007	−005	−008	+002	−067	−061	−068	−067	−063	−072
945	8	−001	000	−005	off	+008	−002	−008	+003	−083	−076	−083	−083	−076	−090
990	9	+005	+006	000		+012	+002	−001	+008	−095	−087	−094	−093	−089	−105
1250	10	+023	+022	+023			+024	+017	+027	−152	−133	−146	−144	−137	−182
1361	11	+049	+049	+045		+046	+037	+029	+049	−204	−181	−198	−205	−185	−253
1440	12	+083	+092	+088		+068	+061	+046	+078	−276	−243	−258	−283	−246	−338
1440	13*											−260			
1616	14	+122	+129	+115		+087	+077	+069	+114	−439	−298	−300	−410	−323	−445
1621.5	15	+176	+182	+146		+089	+085	+091	+153	−556	−346	−279	−705	−373	−500

* Series 13 was read after leaving load on column over night, i. e., 16½ hours.

† Reading Series 9, G. L. 43 C seems questionable; probably should be +011.

COLUMN NO. 1—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.													
		59 C	60 C	61 C	62 C	63 C	64 C	65 C	66 C	67 C	68 C	69 C	70 C	71 C	72 C
242	4	−008	−007	−010	−004	−018	−017	−015	−017	−018	−018	−018	−018	−020	−023
479	5	−029	−036	−034	−021	−047	−044	−055	−041	−043	−046	−043	−049	−043	−053
585	6	−036	−039	−044	−031	−058	−057	−058	−050	−051	−052	−053	−054	−053	−060
822	7	−054	−059	−063	−046	−084	−080	−084	−071	−073	−075	−072	−072	−073	−085
945	8	−068	−075	−076	−058	−106	−098	−103	−088	−089	−097	−083	−093	−089	−106
990	9	−079	−083	−086	−063	−122	−117	−118	−098	−098	−112	−096	−105	−103	−123
1250	10	−123	−133	−139	−106	−213	−189	−193	−146	−148	−210	−139	−157	−147	−177
1361	11	−170	−184	−196	−153	−295	−266	−266		−191	−298	−178	−214	−196	−224
1440	12	−250	−269	−281	−216	−384	−379	−401		−250	−411	−229	−300	−261	−280
1440	13*		−276					−420					−310		
1616	14	−376	−439	−365	−270	−439	−551	−704		−288	−574	−303	−415	−331	−339
1621.5	15	−484	−763	−458	−308	−404	−646	Plug loose		−280	−663	−341	−531	−511	−344

* Series 13 was read after leaving load on column over night, i. e., 16½ hours.

COLUMN NO. 1 A.

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		1 C	2 S	3 C	4 S	5 C	6 S	7 C	8 S	9a C	9 S	10a C	10 S	11a C	11 S	12a C
400	3	+003	+009	+002	+002	+003	+003	+003	+006	+001	+002	+004	+002	000	+003	+002
800	4	+005	+013	+004	+006	+016	+009	+008	+013	+011	+012	+013	+008	+004	+007	+013
1100	5	+010	+024	+012	+017	+017	+017	+024	+028	+033	+033	+030	+029	+012	+016	+037
1200	6	+016	+029	+023	+024	+026	+028	+035	+031	+050	+048	+048	+044	+027	+038	+057
1551	7	+122	+218	+138	+143	+185	+186	+171	+287	+650	+605	+641	+584	+510	+611	+496

COLUMN NO. 1 A—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		12 S	13 C	14 S	15 C	16 S	17 C	18 S	19 C	20 S	21a C	21 S	22a C	22 S	23a C	23 S
400	3	+002	+006	+009	+003	+003	000	+001	+002	−003	−002	−005	+003	+003	Holes	000
800	4	+015	+018	+023	+009	+008	+005	+004	+008	+003	+008	+003	+011	+010	for	+003
1100	5	+035	+040	+039	+023	+020	+008	+013	+029	+016	+028	+017	+029	+029	gage	+011
1200	6	+054	+057	+055	+035	+033	+018	+023	+046	+025	+040	+026	+048	+043	line not	+022
1551	7	+453	+258	+227	+261	+233	+193	+211	+190	+165	+140	+112	+177	+175	drilled	+148

Total Load, in Thousands of Pounds.	Series.	Lateral.										Longitudinal.				
		24a C	24 S	25 C	26 S	27 C	28 S	29 C	30 S	31 C	32 S	41 C	43 C	45 C	47 C	49 C
400	3	+002	+004	−001	+003	000	+003	+010	+003	+002	000	−015	−014	−013	Holes	−018
800	4	+004	+006	+003	+008	+006	+008	+018	+009	+006	+007	−041	−031	−036	for	−058
1100	5	+012	+011	+015	+016	+014	+017	+021	+012	+013	+012	−100	−054	−085	gage	−113
1200	6	+020	+019	+033	+023	+025	+016	+023	+018	+016	+017	−133	−069	−114	line not	−151
1551	7	+108	+100	+090	+059	+048	+042	+034	+023	+044	+046	−676	−193	−453	drilled	−945

Total Load, in Thousands of Pounds.	Series.	Longitudinal.														
		51 C	53 C	55 C	57 C	59 C	61 C	63 C	65 C	67 C	69 C	71 C	73 C	75 C	77 C	79 C
400	3	−019	−010	−014	−025	−019	Holes	−018	−023	−015	−018	−019	−021	−019	−019	−025
800	4	−050	−036	−0.4	−069	−051	for	−051	−070	−053	−048	−048	−040	−048	−040	−056
1100	5	−074	−080	−121	−120	−088	gage	−105	−100	−098	−090	−084	−104	−081	−071	−084
1200	6	−135	−115	−164	−149	−111	line not	−143	−121	−129	−116	−101	−123	−100	−089	−108
1551	7	−1186	−768	−620	−403	−486	drilled	−334	−221	−351	−323	−185	−230	−165	−136	−200

COLUMN NO. 1 B.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.												
		1 C	2 S	3 C	4 S	5 C	6 S	7 C	8 S	9 S	10 S	11 S	12 S	13 C
400	3	+001	+003	+003	+003	+003	+003	+004	+003	+006	+004	+002	+006	+004
800	4	+007	+008	+008	+004	+012	+011	+012	+009	+013	+009	+008	+010	+015
950	5	+009	+009	+014	+008	+016	+012	+018	+013	+017	+014	+014	+018	+016
1100	6	+013	+013	+020	+012	+028	+022	+026	+025	+023	+018	+018	+021	+029
1250	7	+021	+023	+033	+020	+051	+038	+042	+038	+029	+027	+028	+030	+040
1450	8	+043	+041	+064	+050	+112	+084	+096	+082	+059	+066	+057	+066	+085
1700	9	+165	+156	+203	+171	+158	+258	+278	+240	+198	+179	+178	+213	+243
1755	10	+283	+239	+448	+293	+529	+431	+441	+368	+419	+334	+353	+443	+245

Total Load, in Thousands of Pounds.	Series.	Lateral.												
		14 S	15 C	16 S	17 C	18 S	19 C	20 S	21 S	22 S	23 S	24 S	25 C	26 S
400	3	+005	+007	+003	+005	+004	+004	+005	+005	+003	+001	+002	+006	+006
800	4	+010	+013	+012	+011	+009	+009	+013	+012	+013	+003	+008	+018	+015
950	5	+014	+020	+014	+015	+015	+016	+017	+013	+016	+008	+010	+024	+022
1100	6	+019	+026	+021	+019	+018	+019	+028	+018	+018	+008	+013	+033	+028
1250	7	+031	+033	+028	+030	+033	+036	+037	+029	+028	+013	+024	+052	+049
1450	8	+070	+079	+064	+058	+057	+070	+072	+035	+037	+018	+030	+089	+081
1700	9	+212	+230	+185	+192	+174	+221	+201	+078	+075	+063	+077	+232	+191
1755	10	+313	+332	+271	+248	+214	+287	+265	+108	+131	+103	+104	+308	+257

COLUMN NO. 1 B—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.						Longitudinal.						
		27 C	28 S	29 C	30 S	31 C	32 S	41 C	43 C	45 C	47 C	49 C	51 C	53 C
400	3	+003	+006	+003	+002	+004	+002	−023	−024	−024	−024	−028	−028	−021
800	4	+011	+013	+007	+003	+013	+010	−048	−055	−066	−060	−059	−059	−059
950	5	+017	+016	+010	+008	+018	+012	−058	−071	−086	−080	−065	−071	−075
1100	6	+021	+024	+010	+011	+023	+014	−073	−091	−113	−101	−083	−091	−096
1250	7	+038	+035	+018	+018	+039	+028	−089	−118	−149	−130	−103	−115	−121
1450	8	+068	+064	+032	+037	+070	+053	−130	−190	−246	−249	−149	−173	−185
1700	9	+196	+157	+119	+108	+162	+131	−286	−523		−440	−365	−391	−348
1755	10	+283	+215	+165	+151	+209	+169	−470			−280		−596	−530

Series 9: crack near G. L. 45.

Series 10: cracks near G. L. 43, 47, 49, 55 prevented reading satisfactorily.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.												
		55 C	57 C	59 C	61 C	63 C	65 C	67 C	69 C	71 C	73 C	75 C	77 C	79 C
400	3	−024	−029	−028	−023	−024	Missed	−023	−015	−019	−028	−023	−006	−024
800	4	−059	−061	−054	−055	−058	−070	−056	−045	−051	−073	−063	−034	−060
950	5	−068	−078	−069	−074	−074	−091	−069	−053	−063	−083	−080	−041	−074
1100	6	−090	−095	−086	−090	−091	−118	−084	−069	−080	−108	−100	−055	−089
1250	7	−118	−119	−106	−113	−110	−121	−101	−084	−096	−128	−125	−069	−115
1450	8	−189	−180	−159	−156	−155	−160	−136	−116	−131	−184	−176	−090	−140
1700	9	−445	−384	−323	−296	−293	−263	−275	−234	−240	−310	−320	−184	−323
1755	10		−535	−505	−370	−363	−340	−405	−323	−288	*	−460	−233	−425

* Series 10: G. L. 73 C, reading beyond range of instrument as set.

COLUMN NO. 2.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		1 C	2 S	3 C	4 S	5 C	6 S	7 C	8 S	9 S	10 S	11 S	11a C	12 S	13 C	14 S
450	3	+002	000	−005	+008	−003	−002	+003	+002	+002	+004	+002	+011	+007	+003	+004
900	4	+004	+013	+004	+011	+007	+005	+011	+008	+003	+011	+003	+016	+015	+017	+011
1000	5														+012	+013
1200	6	+022	+014	+021	+022	+020	+018	+020	+022	+015	+017	+015	+037	+033	+033	+025
1400	7	+041	+038	+043	+050	+039	+033	+030	+028	+057	+048	+039	+069	+067	+058	+049
1500	8	+063	+058	+068	+066	+069	+054	+049	+035	+092	+088	+072	+118	+111	+090	+081
1600	9	+089	+085	+137	+125	+106	+103	+063	+048	+169	+139	+161	+220	+220	+168	+141

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		15 C	16 S	17 C	18 S	19 C	20 S	21 S	22 S	23 S	23a C	24 S	25 C	26 S	27 C	28 S
450	3	+007	+004	+006	+004	+001	+002	+003	000	+005	+003	000	+007	+005	+004	+003
900	4	+008	+006	+012	+005	+006	+003	+009	+006	+009	+008	+008	+012	−003	+008	+007
1000	5	+017	+013	+013	+010	+017	+009									
1200	6	+021	+015	+019	+008	+018	+016	+021	+013	+013	+016	+021	+023	+013	+018	+018
1400	7	+037	+028	+043	+033	+043	+033	+043	+034	+023	+026	+041	+038	+045	+045	+029
1500	8	+070	+055	+078	+068	+078	+062	+077	+081	+058	+056	+081	+042	+083	+069	+043
1600	9	+120	+101	+153	+133	+161	+130	+146	+163	+104	+113	+157	+059	+118	+123	+078

COLUMN NO. 2—(*Continued*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.				Longitudinal.										
		29 C	30 S	31 C	32 S	41 C	42 S	43 C	44 S	45 C	46 S	47 C	48 S	49 C	50 S	51 C
450	3	+003	−001	+005	+001	−029	−028	−018	−020	−016	−020	−024	−025	−032	−021	−019
900	4	+008	+003	+010	+008	−059	−065	−056	−054	−044	−055	−056	−060	−074	−061	−055
1000	5															
1200	6	+023	+019	+030	+018	−094	−103	−098	−091	−074	−095	−091	−095	−123	−104	−089
1400	7	+050	+040	+056	+031	−124	−163	−151	−151	−119	−149	−138	−123	−168	−159	−136
1500	8	+082	+070	+085	+051	−154	−210	−198	−210	−160	−186	−181	−151	−231	−230	−189
1600	9	+135	+123	+122	+080	−188	−283	−298	−336	−254	−260	−210	−169	−340	−335	−235

Total Load, in Thousands of Pounds.	Series.	Longitudinal.														
		52 S	53 C	54 S	55 C	56 S	57 C	58 S	59 C	60 S	61 C	62 S	63 C	64 S	65 C	66 S
450	3	−020	−019	−019	−023	−023	−035	−030	−022	−018	−015	−025	−026	−025	−029	−034
900	4	−053	−054	−054	−065	−054	−079	−070	−044	−051	−055	−060	−068	−066	−066	−078
1000	5						−110	−091	−062	−055	−074	−074	−089	−080		
1200	6	−088	−088	−086	−108	−110	−136	−125	−073	−081	−100	−099	−120	−125	−114	−125
1400	7	−129	−133	−149	−170	−174	−202	−210	−104	−116	−170	−162	−184	−195	−182	−191
1500	8	−179	−193	−238	−249	−248	−381	−289	−135	−161	−246	−219	−264	−276	−275	−278
1600	9	−216	−315	−390	−474	−458	−440	−441	−181	−211	−406	−334	−435	−439	−478	−452

COLUMN No. 2—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.													
		67 C	68 S	69 C	70 S	71 C	72 S	73 C	74 S	75 C	76 S	77 C	78 S	79 C	80 S
450	3	−020	−026	−018	−022	−026	−035	−030	−030	−029	−023	−019	−016	−030	−034
900	4	−056	−058	−051	−054	−065	−073	−073	−066	−074	−060	−056	−054	−081	−073
1000	5														
1200	6	−088	−085	−080	−086	−105	−111	−123	−130	−138	−100	−094	−091	−135	−115
1400	7	−130	−129	−127	−135	−142	−154	−176	−214	−235	−169	−153	−161	−201	−171
1500	8	−200	−191	−195	−194	−191	−200	−225	−284	−355	−264	−229	−248	−275	−209
1600	9	−358	−329	−291	−268	−269	−295	−288	−374	−608	−453	−356	−388	−360	−263

COLUMN NO. 2 A.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.															
		1 C	2 S	3 C	4 S	5 C	6 S	7 C	8 S	9a C	9 S	10a C	10 S	11a C	11 S	12a C	12 S
400	3	+004	+002	+003	−007	+006	+003	+003	+001	+003	+004	+003	+003	+007	+003	+006	+001
800	4	+008	+003	+004	000	+015	+005	+005	+001	+007	+008	+010	+008	+013	+008	+011	+004
950	5	+013	+009	+013	+001	+021	+008	+009	+004	+008	+014	+017	+012	+021	+011	+015	+006
1150	6	+023	+011	+021	+003	+038	+018	+018	+008	+018	+018	+023	+017	+030	+017	+023	+011
1250	7	+032	+013	+025	+009	+050	+027	+025	+013	+025	+025	+032	+022	+035	+020	+027	+013
1400	8	+058	+033	+045	+024	+086	+047	+052	+029	+041	+041	+052	+040	+058	+032	+046	+029
1500	9	+096	+056	+070	+046	+127	+070	+081	+048	+071	+069	+085	+073	+091	+053	+079	+050
1635	10	+158	+105	+128	+096	+201	+121	+148	+088	+148	+143	+170	+141	+174	+113	+166	+119
1560	11	+238	+166	+213	+168	+266	+166	+194	+137	+228	+219	+247	+167	+252	+169	+245	+172

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		13 C	14 S	15 C	16 S	17 C	18 S	19 C	20 S	21a C	21 S	22a C	22 S	23a C	23 S	24 S
400	3	+005	+003	+009	+003	+007	+013	+003	+005	+010	+003	+008	000	+007	+003	+003
800	4	+010	000	+019	+011	+013	+012	+005	+008	+009	+005	+010	−003	+013	+007	+002
950	5	+012	+006	+024	+018	+021	+018	+015	+012	+015	+006	+017	+005	+020	+013	+005
1150	6	+020	+009	+049	+033	+045	+037	+025	+022	+023	+007	+025	+006	+028	+023	+007
1250	7	+037	+023	+074	+048	+068	+055	+038	+031	+025	+013	+033	+016	+036	+032	+011
1400	8	+095	+070	+139	+094	+123	+088	+098	+081	+043	+023	+055	+028	+059	+048	+022
1500	9	+163	+118	+208	+143	+184	+151	+163	+128	+073	+049	+087	+059	+093	+073	+040
1635	10	+284	+215	+347	+248	+332	+288	+302	+250	+154	+118	+163	+117	+185	+137	+095
1560	11									+251	+209	+276		+319	+187	+153

COLUMN NO. 2 A—(*Continued*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.								Longitudinal.							
		25 C	26 S	27 C	28 S	29 C	30 S	31 C	32 S	41 C	42 S	43 C	44 S	45 C	46 S	47 C	48 S
400	3	+003	+004	+003	+005	+008	+011	+006	+004	−021	−020	−020	−019	−028	−024	−025	−020
800	4	+005	+004	+006	+008	+014	+014	+012	+007	−058	−055	−065	−059	−075	−059	−065	−049
950	5	+008	+010	+019	+015	+026	+018	+016	+008	−075	−068	−084	−075	−091	−078	−079	−058
1150	6	+008	+010	+031	+019	+040	+031	+019	+011	−105	−090	−116	−100	−124	−101	−109	−086
1250	7	+013	+017	+048	+033	+053	+042	+025	+019	−131	−121	−140	−121	−154	−116	−125	−111
1400	8	+023	+024	+079	+054	+087	+058	+048	+039	−166	−179	−194	−174	−208	−163	−191	−170
1500	9	+046	+049	+114	+078	+118	+086	+082	+069	−256	−224	−250	−228	−276	−219	−260	−238
1635	10	+100	+108	+198	+138	+185	+154	+159	+138	−381	−314	−384	−351	−418	−330	−373	−346
1560	11	+151	+158	+293	+208	+257	+222	+221	+196	−524	−435	−554	−508	−515	−413	−421	−391

Total Load, in Thousands of Pounds.	Series.	Longitudinal.															
		49 C	50 S	51 C	52 S	53 C	54 S	55 C	56 S	57 C	58 S	59 C	60 S	61 C	62 S	63 C	64 S
400	3	−018	−024	−019	−016	−016	−020	−015	−019	−016	−019	−014	Gage	−023	−016	−016	−016
800	4	−048	−058	−065	−055	−048	−049	−046	−063	−049	−054	−046	line	−069	−068	−049	−050
950	5	−053	−073	−086	−065	−058	−064	−056	−069	−061	−068	−059	not	−093	−103	−065	−064
1150	6	−081	−100	−119	−095	−076	−085	−079	−093	−084	−095	−086	laid	−141	−156	−086	−090
1250	7	−100	−120	−146	−108	−090	−095	−095	−116	−101	−113	−111	off	−184	−209	−106	−111
1400	8	−131	−161	−201	−163	−113	−115	−133	−143	−144	−133	−159		−290	−339	−161	−189
1500	9	−165	−205	−285	−239	−141	−151	−178	−179	−195	−191	−223		−415	−473	−224	−266
1635	10	−263	−338	−459	−393	−191	−246	−269	−282	−284	−259	−325		−829	−869	−385	−481
1560	11	−403	−510	−646	−540	−239	−319	−336	−356	−450	−486	−424				−904	

COLUMN NO. 2 A—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.															
		65 C	66 S	67 C	68 S	69 C	70 S	71 C	72 S	73 C	74 S	75 C	76 S	77 C	78 S	79 C	80 S
400....	3	−015	−014	−005	−014	−025	−021	−018	−016	−020	−018	−025	−020	−016	−024	−023	−021
800....	4	−046	−049	−049	−044	−066	−055	−054	−050	−050	−053	−071	−060	−041	−063	−059	−053
950....	5	−061	−066	−074	−065	−089	−073	−070	−065	−068	−068	−090	−081	−053	−079	−074	−063
1150....	6	−084	−089	−098	−091	−115	−099	−094	−084	−085	−088	−128	−123	−069	−091	−103	−086
1250....	7	−105	−109	−123	−105	−144	−129	−108	−103	−105	−106	−160	−154	−081	−098	−124	−109
1400....	8	−149	−173	−178	−131	−201	−210	−155	−149	−150	−148	−218	−204	−105	−119	−178	−176
1500....	9	−228	−261	−258	−185	−278	−294	−224	−224	−209	−225	−289	−263	−135	−146	−249	−250
1635....	10	−423	−491	−445	−314	−443	−454	−371	−351	−353	−380	−454	−395	−200	−240	−421	−403
1560....	11	−709	−832	−789	−498	−571	−564	−494	−469	−463	−503	−656	−534	−264	−334	−528	−495

COLUMN NO. 2 B.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		1 C	2 S	3 C	4 S	5 C	6 S	7 C	8 S	9 S	10 S	11 S	12 S	13 C	14 S	15 C
400	3	+002	+003	+006	+007	+005	+003	+004	+003	+003	−001	+003	+008	+004	+003	+002
800	4	+006	+007	+012	+011	+011	+002	+006	+010	+009	+003	+008	+013	+008	+010	+016
1000	5	+009	+011	+017	+014	+015	+008	+012	+011	+014	+011	+014	+022	+011	+013	+023
1150	6	+018	+015	+023	+018	+018	+015	+015	+017	+017	+018	+021	+028	+015	+018	+029
1300	7	+025	+022	+033	+029	+032	+028	+019	+021	+028	+032	+037	+040	+018	+026	+038
1500	8	+050	+040	+063	+054	+063	+053	+037	+036	+078	+079	+090	+093	+043	+048	+067
1650	9	+121	+093	+123	+109	+133	+121	+070	+075	+167	+164	+173	+182	+093	+092	+128
1750	10	+185	+143	+173	+158	+196	+174	+102	+108	+261	+245	+264	+274	+129	+137	+181

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		16 S	17 C	18 S	19 C	20 S	21 S	22 S	23 S	24 S	25 C	26 S	27 C	28 S	29 C	30 S
400	3	+003	+005	+014	+007	+003	+003	+003	+006	+003	+006	+004	+004	+003	+005	+005
800	4	+008	+010	+012	+013	+001	+011	+011	+007	+008	+012	+006	+008	+006	+008	+008
1000	5	+012	+017	+019	+020	+013	+020	+012	+011	+012	+014	+012	+012	+009	+013	+013
1150	6	+018	+024	+023	+027	+018	+027	+015	+018	+015	+021	+017	+019	+013	+019	+016
1300	7	+024	+032	+029	+035	+026	+028	+017	+021	+021	+026	+027	+018	+017	+023	+018
1500	8	+053	+060	+053	+075	+053	+033	+024	+033	+028	+053	+043	+038	+033	+040	+036
1650	9	+103	+118	+108	+144	+110	+052	+043	+060	+039	+111	+094	+093	+081	+073	+066
1750	10	+153	+174	+153	+203	+150	+072	+061	+084	+056	+154	+129	+137	+118	+098	+083

COLUMN No. 2 B—(*Continued*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series	Lateral.		Longitudinal.											
		31 C	32 S	41 C	42 S	43 C	44 S	45 C	46 S	47 C	48 S	49 C	50 S	51 C	52 S
400	3	+010	+008	−025	−021	−021	−026	−029	−026	−029	−020	−033	−028	−024	−030
800	4	+013	+012	−066	−051	−045	−056	−065	−059	−069	−051	−075	−061	−059	−064
1000	5	+020	+019	−095	−074	−060	−091	−086	−080	−093	−073	−103	−085	−084	−085
1150	6	+026	+023	−114	−091	−069	−096	−105	−096	−109	−083	−124	−100	−101	−105
1300	7	+031	+034	−148	−123	−076	−119	−134	−126	−135	−103	−171	−143	−136	−134
1500	8	+060	+064	−254	−218	−099	−174	−220	−230	−219	−191	−308	−290	−226	−225
1650	9	+102	+118	−464	−371	−123	−245	−368	−378	−364	−323	Missed	−493	−394	−401
1750	10	+133	+154	−671	−491	−155	−318	−499	−505	−491	−434	−868	−739	−575	−589

Total Load, in Thousands of Pounds.	Series.	Longitudinal.													
		53 C	54 S	55 C	56 S	57 C	58 S	59 C	60 S	61 C	62 S	63 C	64 S	65 C	66 S
400	3	−026	−024	−031	−025	−026	−026	−025	−023	−028	−023	−029	−024	−021	−023
800	4	−065	−069	−080	−061	−061	−055	−058	−053	−058	−055	−066	−061	−043	−046
1000	5	−094	−089	−105	−080	−080	−076	−081	−074	−084	−079	−091	−080	−056	−060
1150	6	−118	−108	−131	−096	−098	−086	−099	−089	−101	−094	−109	−093	−059	−066
1300	7	−163	−158	−180	−145	−116	−104	−121	−111	−116	−115	−133	−110	−068	−078
1500	8	−300	−324	−329	−288	−181	−190	−181	−191	−166	−205	−199	−193	−088	−098
1650	9	−539	−585	−599	−481	−320	−325	−295	−295	−271	−328	−321	−308	−113	−119
1750	10	−808	Out of range	−916	−730	−443	−431	−410	−396	−368	−431	−419	−401	−128	−143

COLUMN NO. 2 B—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.													
		67 C	68 S	69 C	70 S	71 C	72 S	73 C	74 S	75 C	76 S	77 C	78 S	79 C	80 S
400	3	−023	−025	−023	−015	−030	−023	−018	−023	−013	−021	−020	−021	−030	−020
800	4	−045	−053	−055	−044	−063	−054	−044	−058	−044	−055	−054	−054	−070	−059
1000	5	−059	−071	−071	−063	−081	−068	−063	−076	−054	−074	−073	−074	−096	−088
1150	6	−064	−078	−084	−076	−094	−080	−070	−086	−064	−088	−093	−090	−119	−108
1300	7	−078	−095	−098	−098	−111	−095	−083	−099	−066	−104	−104	−111	−159	−161
1500	8	−098	−111	−135	−140	−143	−128	−106	−119	−079	−138	−154	−140	−239	−253
1650	9	−125	−134	−173	−176	−199	−175	−146	−173	−111	−195	−228	−196	−360	−389
1750	10	−148	−149	−218	−219	−244	−203	−180	−221	−135	−246	−283	−243	−440	−478

COLUMN No. 3.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		1 C	2 S	3 C	4 S	5 C	6 S	7 C	8 S	9 S	10 S	11 S	12 S	13 C	14 S	15 C
450	3	+004	+003	+003	+002	+002	+003	+003	+002	+003	+004	+003	−001	+005	+004	+002
900	4	+013	+011	+015	+010	+016	+015	+009	+009	+012	+013	+013	+005	+014	+010	+011
1100	5	+029	+020	+029	+018	+033	+028	+018	+018	+016	+016	+019	+013	+023	+018	+021
1300	6	+063	+046	+067	+043	+069	+055	+039	+037	+024	+026	+026	+025	+045	+033	+031
1400	7	+092	+068	+107	+070	+103	+083	+060	+052	+027	+033	+035	+027	+058	+047	+041
1600	8	+248	+178	+283	+182	+211	+195	+153	+142	+095	+111	+118	+099	+150	+128	+122
1685	9	+345	+258	+375	+250	+275	+254	+213	+193	+142	+175	+170	+144	+203	+177	+168

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		16 S	17 C	18 S	19 C	20 S	21 S	22 S	23 S	24 S	25 C	26 S	27 C	28 S	29 C	30 S
450	3	+003	+003	+005	+003	+004	+010	+003	+005	+003	+005	+007	+006	+003	+003	+002
900	4	+010	+009	+013	+013	+016	+015	+007	+010	+008	+012	+013	+010	+008	+009	+008
1100	5	+016	+013	+015	+021	+023	+021	+009	+010	+014	+016	+018	+016	+012	+013	+011
1300	6	+025	+027	+021	+038	+044	+023	+013	+015	+020	+026	+026	+019	+018	+019	+018
1400	7	+033	+034	+028	+062	+065	+025	+014	+018	+024	+032	+030	+025	+020	+022	+019
1600	8	+104	+118	+092	+163	+139	+043	+022	+025	+043	+056	+052	+044	+036	+047	+041
1685	9	+143	+170	+127	+223	+188	+053	+030	+035	+055	+076	+075	+063	+055	+074	+066

COLUMN NO. 3—(*Continued*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.		Longitudinal.											
		31 C	32 S	41 C	42 S	43 C	44 S	45 C	46 S	47 C	48 S	49 C	50 S	51 C	52 S
450	3	+004	+003	−039	−031	−028	−024	−034	−013	−018	−028	−021	−026	−026	−028
900	4	+014	+013	−089	−071	−068	−070	−089	−070	−044	−058	−050	−056	−069	−064
1100	5	+023	+019	−129	−099	−096	−101	−126	−096	−069	−081	−068	−080	−100	−093
1300	6	+021	+029	−194	−173	−139	−166	−195	−158	−098	−105	−089	−103	−128	−125
1400	7	+045	+037	−258	−256	−193	−220	−279	−251	−126	−130	−111	−116	−168	−178
1600	8	+086	+071	−554	−531	−421	−438	−623	−541	−280	−265	−198	−234	−365	−400
1685	9	+118	+090	−751	−689	−550	−550			−411	−368	−261	−344	−511	−536

Total Load, in Thousands of Pounds.	Series.	Longitudinal.													
		53 C	54 S	55 C	56 S	57 C	58 S	59 C	60 S	61 C	62 S	63 C	64 S	65 C	66 S
450	3	−008	−026	−005	−014	−023	−030	−030	−026	−030	−025	−033	−028	−024	−035
900	4	−038	−065	−035	−049	−045	−068	−068	−065	−084	−044	−070	−056	−053	−070
1100	5	−049	−088	−058	−068	−064	−091	−093	−086	−113	−069	−100	−081	−064	−083
1300	6	−063	−113	−061	−091	−073	−115	−120	−106	−148	−088	−141	−111	−075	−103
1400	7	−074	−143	−078	−109	−091	−136	−144	−136	−181	−133	−195	−146	−083	−110
1600	8	−108	−225	−134	−186	−110	−251	−288	−288	−366	−283	−421	−286	−110	−128
1685	9	−154	−318	−175	−246	−136	−331	−379	−383	−486	−379	−580	−393	−128	−143

COLUMN NO. 3—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.													
		67 C	68 S	69 C	70 S	71 C	72 S	73 C	74 S	75 C	76 S	77 C	78 S	79 C	80 S
450	3	−018	−021	−019	−015	−029	−024	−039	−043	−024	−018	−013	−016	−019	−028
900	4	−048	−055	−061	−049	−085	−063	−065	−069	−065	−055	−046	−049	−046	−068
1100	5	−060	−070	−079	−070	−109	−076	−076	−084	−085	−070	−058	−066	−058	−088
1300	6	−070	−079	−100	−085	−141	−099	−096	−096	−103	−089	−075	−085	−073	−108
1400	7	−084	−096	−114	−096	−158	−106	−108	−108	−118	−094	−085	−098	−079	−118
1600	8	−110	−120	−170	−158	−250	−151	−136	−138	−169	−148	−120	−130	−170	−159
1685	9	−135	−139	−219	−218	−318	−186	−156	−155	−214	−183	−153	−163		−191

COLUMN NO. 3 A.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.															
		1 C	2 S	3 C	4 S	5 C	6 S	7 C	8 S	9a C	9 S	10a C	10 S	11a C	11 S	12a C	12 S
400....	3	+003	+002	−005	−001	+003	−001	+003	+008	+003	+003	+004	+003	−003	−003	+006	000
800....	4	+009	+009	+006	+009	+011	+007	+004	+013	+011	+014	+012	+006	+008	000	+014	+007
1000....	5	+016	+018	+012	+024	+018	+008	+009	+013	+019	+018	+023	+007	+012	+003	+018	+013
1200....	6	+033	+038	+028	+050	+036	+023	+021	+023	+033	+033	+043	+030	+028	+016	+027	+023
1400....	7	+076	+093	+063	+112	+095	+077	+055	+054	+099	+096	+115	+085	+085	+083	+088	+091
1600....	8	+183	+208	+227	+288	+263	+238	+159	+156	+247	+245	+288	+238	+201	+198	+208	+260
80....	9	+231	+267				+577		+271					+447	+430	+433	

Series 9 was taken after column was subjected to its ultimate load, 1,647,000 pounds: the testing machine was operated continuously until load sustained had fallen to 1,541,000 pounds without rupture of spirals. Load was then released to 80,000 pounds and Series 9 taken.

Total Load, in Thousands of Pounds.	Series.	Lateral.															
		13 C	14 S	15 C	16 S	17 C	18 S	19 C	20 S	21a C	21 S	22a C	22 S	23a C	23 S	24a C	24 S
400....	3	+003	+002	+001	−001	+006	+003	+005	+003	−009	000	+002	−001	+003	+001	+003	−002
800....	4	+013	+008	+012	+005	+013	+008	+009	+007	+008	+006	+009	+022	+010	+017	+011	+003
1000....	5	+017	+015	+018	+012	+019	+012	+016	+009	+010	+006	+012	+022	+014	+003	+012	000
1200....	6	+028	+025	+028	+024	+035	+021	+028	+023	+012	+005	+013	+023	+026	+013	+015	+009
1400....	7	+079	+074	+073	+067	+084	+064	+094	+073	+017	+008	+021	+027	+044	+022	+034	+030
1600....	8	+203	+196	+174	+168	+199	+166	+217	+182	+041	+026	+058	+069	+104	+084	+078	+076
80....	9	+179	+174	+156	+153	+193	+152	+208	+173	+033	+023	+055	+056	+093	+063	+073	+059

COLUMN NO. 3 A—(*Continued*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.								Longitudinal.							
		25 C	26 S	27 C	28 S	29 C	30 S	31 C	32 S	41 C	42 S	43 C	44 S	45 C	46 S	47 C	48 S
400....	3	+004	000	+006	+002	−002	+004	+005	+003	−018	−028	−034	−029	−023	−024	−024	−019
800....	4	+007	+003	+013	+005	+012	+004	+008	+010	−061	−066	−075	−073	−048	−055	−050	−051
1000....	5	+008	+006	+019	+018	+019	+023	+017	+019	−088	−099	−098	−108	−068	−079	−071	−070
1200....	6	+011	+003	+020	+023	+030	+032	+027	+024	−139	−128	−120	−141	−099	−118	−100	−094
1400....	7	+024	+018	+042	+031	+063	+062	+056	+050	−255	−239	−166	−245	−178	−196	−179	−125
1600....	8	+077	+063	+097	+092	+139	+135	+123	+113	−426	−459	−258	−510	−475	−486	−405	−329
80....	9	+061	+041	+084	+078	+121	+109	+101	+085	−440	−576	−323				−824	−765

Total Load, in Thousands of Pounds.	Series.	Longitudinal.															
		49 C	50 S	51 C	52 S	53 C	54 S	55 C	56 S	57 C	58 S	59 C	60 S	61 C	62 S	63 C	64 S
400....	3	−033	−029	−040	−029	−019	−024	−025	−019	−025	−026	−026	−023	−025	−023	−025	−026
800....	4	−078	−066	−101	−074	−059	−061	−060	−090	−063	−063	−053	−080	−059	−056	−061	−061
1000....	5	−109	−096	−146	−106	−078	−086	−084	−111	−086	−086	−063	−083	−083	−080	−085	−080
1200....	6	−161	−133	−241	−199	−111	−129	−121	−141	−123	−114	−085	−113	−111	−104	−118	−118
1400....	7	−356	−264	−510	−420	−186	−209	−245	−178	−216	−148	−121	−169	−171	−149	−226	−253
1600....	8	−729	−573	−1049	−763	−360	−435	−489	−479	−398	−310	−198	−280	−279	−255	−441	−541
80....	9					−586	−736	−808	−900	−315	−238	−140	−201	−225	−206	−365	−520

COLUMN NO. 3 A—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.															
		65 C	66 S	67 C	68 S	69 C	70 S	71 C	72 S	73 C	74 S	75 C	76 S	77 C	78 S	79 C	80 S
400....	3	−024	−026	−010	−026	Gage	−029	−028	−043	−026	−025	−023	−021	−029	−033	−030	−033
800....	4	−049	−051	−050	−055	line	−064	−056	−066	−058	−046	−040	−049	−061	−064	−060	−065
1000....	5	−066	−068	−080	−073	not	−089	−078	−080	−078	−069	−056	−073	−091	−099	−084	−085
1200....	6	−081	−081	−103	−093	laid	−124	−109	−109	−095	−088	−065	−093	−123	−123	−121	−130
1400....	7	−101	−100	−146	−115	off	−189	−166	−176	−133	−113	−081	−116	−186	−189	−176	−196
1600....	8	−150	−155	−286	−198		−394	−313	−329	−235	−214	−110	−184	−316	−311	−274	−309
80....	9			−183	−111		−318	−229	−249	−133	−134	−051	−104	−231	−239	−179	−226

COLUMN NO. 3 B.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		1 C	2 S	3 C	4 S	5 C	6 S	7 C	8 S	9 S	10 S	11 S	12 S	13 C	14 S	15 C
400	3	+008	+004	+006	+003	+005	+004	+004	+003	+007	+006	+003	+003	+005	000	+001
72	4	+003	−002	000	−004	+001	000	−001	−001	+002	+003	000	+003	000	−001	−002
400	5	+009	+003	+008	+006	+006	+003	+005	+013	+011	+010	+008	+007	+007	+002	+005
800	6	+013	+006	+012	+010	+011	+007	+008	+013	+013	+013	+010	+012	+013	+004	+008
70	7	+008	+001	+006	+004	+007	+006	+003	+010	+010	+010	+008	+005	+007	+001	+005
800	8	+014	+006	+013	+010	+013	+008	+010	+013	+013	+012	+010	+010	+013	+001	+008
1200	9	+024	+019	+028	+018	+023	+018	+021	+018	+021	+026	+018	+023	+022	+011	+015
1450	10	+036	+025	+049	+036	+045	+035	+033	+028	+028	+031	+028	+028	+034	+021	+022
70	11	+022	+012	+025	+015	+025	+015	+013	+015	+015	+016	+012	+016	+016	+007	+008
1650	12	+071	+051	+094	+071	+083	+060	+063	+055	+045	+060	+047	+043	+060	+048	+048
2017	13	Plug loose	Plug loose	+522	+443	+276	+251	+314	+223	+188	+252	+233	+204	+204	+188	+183

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		16 S	17 C	18 S	19 C	20 S	21 S	22 S	23 S	24 S	25 C	26 S	27 C	28 S	29 C	30 S
400	3	+003	+003	+002	+005	+003	+001	+003	000	+003	+008	+006	+004	+004	+005	+004
72	4	+001	−001	−002	+001	−002	000	000	−001	000	+004	−001	+003	+001	000	+001
400	5	+003	+004	+003	+007	+007	+004	+003	+003	+004	+008	+008	+006	+007	+002	+002
800	6	+006	+008	+008	+010	+012	+011	+010	+006	+004	+018	+013	+007	+009	+012	+011
70	7	+003	+001	+003	+005	+005	+002	+003	+003	+002	+007	+002	+001	+006	+003	
800	8	+007	+010	+012	+013	+018	+013	+008	+009	+010	+020	+011	+016	+010	+013	+015
1200	9	+011	+018	+017	+023	+022	+018	+017	+013	+017	+031	+018	+014	+014	+018	+018
1450	10	+018	+029	+030	+038	+038	+029	+022	+021	+025	+053	+033	+023	+019	+028	+025
70	11	+007	+012	+009	+017	+015	+012	+008	+006	+008	+031	+018	+011	+010	+008	+012
1650	12	+034	+059	+060	+075	+064	+044	+033	+028	+040	+101	+074	+048	+035	+049	+039
2017	13	+190	+379	+373	+379	+338	+207	+189	+159	+223	+479	+425	+355	+298	+270	+205

COLUMN NO. 3 B—(*Continued*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.		Longitudinal.											
		31 C	32 S	41 C	42 S	43 C	44 S	45 C	46 S	47 C	48 S	49 C	50 S	51 C	52 S
400	3	+001	+003	−016	−023	−031	−020	−021	−023	−021	+008	−024	−023	−019	−023
72	4	−001	+003	−001	−001	−003	+001	+001	000	−001	+029	−004	−001	+001	−003
400	5	000	+003	−015	−019	−026	−021	−018	−019	−019	+016	−021	−020	−016	−023
800	6	+005	+009	−039	−048	−069	−051	−049	−055	−050	−016	−048	−046	−046	−051
70	7	+001	+004	−004	−009	−015	−006	−005	−005	−009	+026	−006	−005	−003	−008
800	8	+005	+012	−040	−050	−070	−058	−050	−056	−053	−019	−048	−048	−049	−050
1200	9	+011	+019	−066	−081	−121	−101	−089	−100	−094	−061	−080	−081	−086	−086
1450	10	+028	+034	−089	−103	−190	−169	−143	−149	−136	−110	−101	−113	−116	−118
70	11	+008	+016	−018	−011	−061	−066	−038	−040	−036	−018	−016	−025	−016	−023
1650	12	+078	+066	−133	−158	−294	−294	−193	−229	−211	−216	−135	−165	−165	−148
2017	13	+307	+253	−466	−688			−381	−484	−700	−720	−230	−330	−474	−448

Total Load, in Thousands of Pounds.	Series.	Longitudinal.													
		53 C	54 S	55 C	56 S	57 C	58 S	59 C	60 S	61 C	62 S	63 C	64 S	65 C	66 S
400	3	−023	Gage	−020	−020	−019	−021	−036	−024	−020	−024	−029	−018	−024	−029
72	4	−001	line	+003	−001	000	−005	−004	+001	000	−006	−011	+001	+003	−004
400	5	−021	not	−015	−018	−018	−025	−028	−023	−020	−025	−030	−016	−022	−029
800	6	−051	laid	−045	−045	−040	−049	−061	−051	−056	−051	−055	−045	−055	−059
70	7	−006	off	+004	−003	+001	000	−006	−005	−005	−006	−013	000	−006	−005
800	8	−051		−045	−046	−039	−048	−060	−053	−059	−053	−058	−049	−060	−060
1200	9	−091		−081	−080	−066	−081	−099	−085	−104	−093	−094	−085	−103	−096
1450	10	−120		−108	−103	−085	−120	−134	−115	−149	−135	−128	−129	−146	−150
70	11	−024		−013	−015	−013	−028	−028	−025	−039	−040	−040	−036	−044	−053
1650	12	−171		−153	−126	−114	−173	−201	−193	−249	−250	−178	−189	−216	−211
2017	13	−690		−345	−359	−225	−339	−633	−584	Out of range	Out of range	−669	−673	−770	−649

COLUMN NO. 3 B—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.													
		67 C	68 S	69 C	70 S	71 C	72 S	73 C	74 S	75 C	76 S	77 C	78 S	79 C	80 S
400	3	−026	−020	−016	−016	−021	−023	−025	−029	−019	−025	−015	−015	−006	−018
72	4	−003	+003	+001	000	000	000	−004	−005	−004	−004	000	000	−001	000
400	5	−028	−021	−015	−018	−019	−020	−025	−029	−018	−025	−014	−014	−013	−019
800	6	−055	−044	−044	−044	−044	−049	−050	−059	−041	−049	−036	−036	−040	−050
70	7	−006	−003	−003	−005	−004	−019	−008	−009	−008	−010	−005	−004	−001	−006
800	8	−058	−050	−045	−044	−045	−050	−005	−061	−040	−050	−036	−038	−049	−051
1200	9	−094	−083	−081	−076	−078	−088	−088	−098	−069	−078	−064	−066	−084	−089
1450	10	−126	−116	−106	−101	−103	−118	−123	−111	−091	−098	−084	−086	−101	−130
70	11	−029	−025	−015	−014	−016	−024	−029	−014	−019	−009	−010	−010	−034	−023
1650	12	−183	−165	−150	−150	−130	−170	−170	−153	−125	−119	−111	−111	−194	−218
2017	13	−620	−563	−598	−611	−380	−526	−408	−664	−389	−405	−218	−251	−545	−646

COLUMN NO. 4.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		1 C	2 S	3 C	4 S	5 C	6 S	7 C	8 S	9 S	10 S	11 S	12 S	13 C	14 S	15 C
450	3	+004	−002	+003	+008	+003	+005	+004	000	+006	+005	+003	000	+006	+005	+003
900	4	+008	−004	+012	+010	+009	+008	+010	+006	+008	+008	+005	+006	+010	+007	+007
1100	5	+013	−001	+020	+017	+015	+013	+016	+010	+013	+013	+008	+002	+011	+011	+014
1300	6	+026	+004	+033	+025	+023	+020	+020	+019	+018	+016	+012	+013	+021	+016	+018
1500	7	+043	+019	+054	+041	+035	+035	+033	+035	+020	+022	+017	+015	+031	+028	+023
1700	8	+085	+057	+100	+077	+078	+077	+072	+078	+030	+032	+034	+028	+052	+039	+039
1775	9	+105	+072	+129	+094	+102	+098	+090	+098	+042	+045	+048	+035	+068	+053	+051
1900	10	+168	+128	+208	+146	+151	+155	+143	+155	+089	+108	+095	+087	+107	+089	+111
2044	11	+488	+358	+408	+294		+366	+444	+457	+231	+351	+342	+279	+252	+193	+234

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		16 S	17 C	18 S	19 C	20 S	21 S	22 S	23 S	24 S	25 C	26 S	27 C	28 S	29 C	30 S
450	3	+004	+007	+006	−003	+008	+002	+007	000	+003	000	+008	+006	+004	+004	+003
900	4	+009	+014	+008	+002	+008	+003	+013	000	+007	+008	+008	+013	+004	+005	+003
1100	5	+015	+016	+013	+009	+016	+013	+015	+007	+013	+015	+013	+018	+009	+013	+008
1300	6	+018	+018	+017	+015	+021	+014	+016	+010	+018	+019	+023	+020	+011	+013	+010
1500	7	+021	+023	+019	+020	+023	+017	+027	+015	+020	+028	+024	+023	+013	+017	+013
1700	8	+036	+033	+031	+039	+037	+041	+041	+018	+038	+035	+033	+032	+020	+028	+021
1775	9	+047	+039	+038	+050	+045	+051	+048	+023	+042	+041	+040	+036	+023	+033	+025
1900	10	+099	+086	+088	+109	+087	+091	+087	+052	+078	+050	+047	+046	+029	+052	+038
2044	11	+211	+218	+193	+277	+220	+226	+177	+109	+192	+081	+083	+083	+055	+084	+078

COLUMN NO. 4—(*Continued*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.		Longitudinal.											
		31 C	32 S	41 C	42 S	43 C	44 S	45 C	46 S	47 C	48 S	49 C	50 S	51 C	52 S
450	3	+006	+007	−026	−029	−035	−021	−020	−020	−028	−026	−024	−028	−020	−028
900	4	+008	+008	−048	−054	−069	−055	−040	−040	−061	−053	−053	−056	−035	−053
1100	5	+013	+013	−060	−064	−104	−071	−053	−056	−086	−071	−065	−073	−043	−064
1300	6	+016	+016	−085	−080	−121	−085	−066	−070	−124	−094	−083	−094	−056	−081
1500	7	+019	+018	−111	−105	−170	−094	−090	−085	−168	−120	−094	−108	−061	−096
1700	8	+027	+027	−153	−185	−251	−133	−135	−133	−296	−304	−124	−135	−083	−125
1775	9	+030	+029	−179	−235	−316	−180	−168	−181	−365	−390	−153	−160	−095	−153
1900	10	+043	+039	−279	−470	−511	−348	−233	−341	−565	−588	−224	−199	−128	−235
2044	11	+088	+075	−620				−506	−706			−354	−404	−365	−663

Total Load, in Thousands of Pounds.	Series.	Longitudinal.													
		53 C	54 S	55 C	56 S	57 C	58 S	59 C	60 S	61 C	62 S	63 C	64 S	65 C	66 S
450	3	−020	−021	−029	−023	−029	−030	−018	−024	−011	−014	−023	−018	−033	−026
900	4	−049	−045	−049	−046	−058	−056	−040	−050	−034	−038	−049	−016	−056	−058
1100	5	−060	−061	−066	−063	−081	−074	−054	−065	−041	−045	−063	−029	−069	−074
1300	6	−084	−079	−084	−079	−105	−095	−070	−081	−055	−065	−086	−078	−086	−091
1500	7	−105	−094	−104	−093	−126	−115	−078	−095	−066	−078	−106	−169	−098	−108
1700	8	−170	−141	−149	−111	−190	−171	−105	−128	−089	−109	−154	−140	−131	−153
1775	9	−234	−214	−191	−130	−220	−205	−124	−134	−099	−129	−193	−180	−154	−175
1900	10	−441	−440	−336	−198	−325	−293	−210	−189	−160	−246	−313	−284	−229	−254
2044	11				−413	−586	−538	−360	−413	−276	−390		−581	−440	−459

COLUMN NO. 4—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.													
		67 C	68 S	69 C	70 S	71 C	72 S	73 C	74 S	75 C	76 S	77 C	78 S	79 C	80 S
450	3	−024	−013	−005	−009	−014	−023	−030	−028	−010	−016	−013	−010	−026	−036
900	4	−061	−041	−026	−035	−050	−054	−063	−059	−034	−041	−039	−030	−056	−052
1100	5	−078	−056	−034	−045	−061	−065	−079	−070	−041	−058	−055	−044	−073	−065
1300	6	−104	−074	−049	−060	−083	−084	−095	−079	−054	−070	−071	−055	−089	−078
1500	7	−129	−089	−056	−073	−103	−104	−114	−089	−060	−081	−089	−063	−109	−091
1700	8	−189	−126	−069	−090	−141	−149	−134	−095	−080	−101	−135	−079	−140	−118
1775	9	−224	−164	−075	−101	−160	−166	−139	−100	−089	−105	−156	−084	−156	−125
1900	10	−335	−275	−094	−131	−233	−256	−160	−110	−101	−111	−211	−125	−196	−149
2044	11	−595	−471	−126	−188	−461	−488	−208	−145	−134	−140	−314		−289	−218

COLUMN NO. 4 A.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.															
		1 C	2 S	3 C	4 S	5 C	6 S	7 C	8 S	9a C	9 S	10a C	10 S	11a C	11 S	12a C	12 S
400....	3	+002	+002	+002	+002	+005	−001	000	−001	+003	+002	+003	+002	+003	+002	+003	+007
800....	4	+007	+003	+004	+001	+007	−003	+005	+002	+008	+006	+006	+004	+012	+008	+008	+008
1000....	5	+009	+007	+008	+004	+013	+001	+010	+005	+009	+010	+010	+005	+013	+008	+010	+010
1000....	6*	+011	+001	+004	+005	+010	−002	+003	−005	+013	+002	+005	−003	+008	−001	+003	+003
1200....	7	+011	+001	+006	+004	+012	−002	+007	−002	+008	−003	+008	−003	+012	+004	+007	+005
1400....	8	+014	+012	+016	+008	+020	+001	+009	+002	+009	+007	+010	−002	+012	+006	+007	+010
1600....	9	+025	+023	+029	+013	+032	+005	+018	+008	+018	+012	+018	+006	+021	+015	+012	+013
1800....	10	+068	+060	+060	+040	+068	+026	+048	+036	+031	+021	+035	+015	+028	+018	+020	+021
1960....	11	+167	+157	+105	+082	+149	+071	+151	+133	+095	+079	+113	+060	+083	+057	+074	+074

* Series 6 was read after load remained on column over night, i. e., 16 hours

Total Load, in Thousands of Pounds.	Series.	Lateral.															
		13 C	14 S	15 C	16 S	17 C	18 S	19 C	20 S	21a C	21 S	22a C	22 S	23a C	23 S	24a C	24 S
400....	3	+001	+006	+003	+002	+004	+008	+003	+008	+007	+003	+003	+003	+003	−002	+005	+005
800....	4	+004	+007	+003	+005	+004	+006	+007	+007	+008	+002	+006	+007	+008	+002	+006	+009
1000....	5	+006	+008	+004	+007	+009	+011	+011	+008	+016	+005	+009	+010	+013	+002	+013	+013
1000....	6*	−003	+004	000	−001	+006	+004	+003	+004	+007	−005	+009	+004	+011	+004	+008	+010
1200....	7	+001	+005	+003	+005	+007	+003	+006	+005	+009	−001	+008	+006	+014	+006	+008	+009
1400....	8	+005	+007	+005	+008	+012	+009	+010	+010	+012	+002	+011	+010	+031	+015	+019	+014
1600....	9	+010	+009	+018	+017	+024	+012	+015	+010	+022	+009	+019	+021	+055	+038	+035	+032
1800....	10	+027	+029	+059	+050	+065	+042	+033	+033	+083	+072	+091	+098	+115	+103	+082	+084
1960....	11	+115	+118	+188	+175	+198	+125	+123	+113	+233	+225	+282	+310	+293	+265	+190	+183

* Series 6 was read after load remained on column over night, i. e., 16 hours.

COLUMN NO. 4 A—(*Continued*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.								Longitudinal.							
		25 C	26 S	27 C	28 S	29 C	30 S	31 C	32 S	41 C	42 S	43 C	44 S	45 C	46 S	47 C	48 S
400....	3	+004	+002	+003	+003	+003	−001	+002	+003	−016	−019	−016	−020	−014	−015	−020	−019
800....	4	+003	+001	+001	+001	+001	+003	+012	+004	−041	−041	−036	−038	−035	−036	−044	−041
1000....	5	+006	+005	+004	+004	+018	+007	+025	+025	−056	−053	−050	−053	−049	−054	−058	−055
1000....	6*	+001	−002	+003	−002	+015	000	+019	+018	−059	−054	−051	−054	−055	−055	−061	−058
1200....	7	+006	+003	+004	+003	+023	+006	+027	+026	−080	−076	−070	−074	−073	−081	−085	−078
1400....	8	+017	+013	+011	+007	+041	+021	+053	+052	−096	−091	−091	−080	−083	−090	−101	−086
1600....	9	+063	+047	+038	+038	+089	+063	+107	+114	−128	−109	−118	−085	−096	−106	−120	−103
1800....	10	+155	+141	+108	+110	+171	+134	+235	+253	−185	−120	−165	−096	−121	−131	−165	−160
1960....	11				+404	+453	+446			−331	−254	−211	−101	−180	−214	−374	−438

* Series 6 was read after load remained on column over night, i. e., 16 hours.
Series 8, scaling of steel noted at G. L. 42 S.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.															
		49 C	50 S	51 C	52 S	53 C	54 S	55 C	56 S	57 C	58 S	59 C	60 S	61 C	62 S	63 C	64 S
400....	3	−018	−015	−016	−014	−019	−016	−016	−023	−016	−016	−014	−015	−031	−015	−019	−018
800....	4	−031	−040	−039	−034	−046	−043	−038	−045	−040	−036	−035	−041	−059	−045	−039	−043
1000....	5	−045	−051	−051	−045	−061	−053	−050	−044	−048	−048	−045	−050	−071	−059	−045	−053
1000....	6*	−050	−055	−050	−048	−056	−060	−054	−065	−048	−046	−048	−050	−080	−065	−054	−060
1200....	7	−065	−073	−076	−073	−101	−083	−075	−088	−065	−070	−073	−076	−099	−086	−070	−084
1400....	8	−085	−088	−094	−085	−112	−093	−083	−091	−084	−086	−095	−095	−115	−101	−079	−096
1600....	9	−105	−109	−123	−111	−138	−109	−095	−111	−104	−113	−120	−123	−140	−120	−086	−105
1800....	10	−144	−141	−171	−153	−174	−141	−118	−140	−136	−158	−201	−244	−200	−194	−108	−115
1960....	11	−274	−283	−315	−316	−305	−249	−189	−215	−259	−295	−496	−575	−369	−400	−160	−125

* Series 6 was read after load remained on column over night, i. e., 16 hours.

COLUMN No. 4 A—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.															
		65 C	66 S	67 C	68 S	69 C	70 S	71 C	72 S	73 C	74 S	75 C	76 S	77 C	78 S	79 C	80 S
400	3	−020	−016	−014	−013	−015	−015	−015	−020	−016	−015	−019	−014	−026	−024	−020	−028
800	4	−039	−038	−034	−036	−040	−045	−034	−046	−035	−034	−031	−030	−066	−059	−051	−050
1000	5	−053	−048	−040	−046	−054	−061	−043	−059	−043	−043	−039	−039	−098	−074	−069	−085
1000	6*	−054	−049	−049	−050	−063	−073	−054	−069	−043	−046	−040	−044	−105	−076	−076	−071
1200	7	−078	−071	−068	−073	−083	−093	−063	−089	−065	−065	−065	−064	−141	−098	−101	−093
1400	8	−095	−089	−086	−095	−101	−104	−074	−103	−090	−074	−084	−079	−193	−141	−141	−134
1600	9	−121	−110	−114	−123	−140	−148	−093	−119	−135	−085	−121	−093	−283	−268	−231	−224
1800	10	−221	−161	−250	−286	−218	−258	−126	−155	−265	−159	−193	−113	−479	−499	−400	−436
1960	11	−520	−424	−680	−739	−435	−571	−164	−216			−504	−460				

* Series 6 was read after load remained on column over night, i. e., 16 hours.

COLUMN NO. 4 B.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		1 C	2 S	3 C	4 S	5 C	6 S	7 C	8 S	9 S	10 S	11 S	12 S	13 C	14 S	15 C
500	3	+005	+008	+005	+009	+005	+005	+007	−001	+006	+004	+004	+005	+005	+004	+005
1000	4	+018	+013	+008	+012	+010	+009	+015	+008	+012	+007	+009	+012	+013	+012	+008
1400	5	+027	+021	+017	+016	+013	+014	+023	+013	+021	+017	+016	+018	+023	+021	+014
1750	6	+076	+064	+048	+047	+069	+025	+047	+034	+054	+043	+024	+036	+058	+048	+032
1850	7	+111	+088	+082	+075	+096	+036	+067	+046	+080	+069	+035	+055	+089	+073	+055
1965	8	+168	+132	+134	+115	+135	+061	+094	+071	+131	+119	+070	+095	+158	+131	+112
2014	9	+208	+159	+172	+149	+164	+080	+126	+087	+166	+149	+088	+122	+198	+162	+143
2106	10	+252	+200	+229	+197	+211	+122	+165	+114	+238	+213	+118	+171	+284	+238	+193
2146	11	+271	+215	+270	+223	+240	+154	+188	+131	+298	+222	+143	+190	+364	+313	+228

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		16 S	17 C	18 S	19 C	20 S	21 S	22 S	23 S	24 S	25 C	26 S	27 C	28 S	29 C	30 S
500	3	+003	+004	+003	+005	+008	+010	+008	+009	+006	+005	+005	+006	+004	+003	+005
1000	4	+007	+010	+009	+014	+018	+014	+011	+013	+010	+010	+008	+011	+008	+009	+008
1400	5	+017	+018	+015	+025	+023	+019	+017	+021	+017	+013	+008	+020	+012	+018	+018
1750	6	+035	+029	+022	+066	+053	+028	+023	+041	+031	+025	+018	+079	+042	+068	+051
1850	7	+053	+050	+040	+102	+084	+042	+037	+062	+048	+037	+023	+118	+066	+096	+080
1965	8	+109	+097	+084	+163	+142	+085	+071	+104	+083	+081	+067	+195	+123	+166	+148
2014	9	+138	+123	+103	+198	+178	+122	+090	+127	+096	+104	+086	+234	+158	+221	+188
2106	10	+201	+157	+141	+264	+243	+177	+131	+181	+163	+156	+148	+322	+238	+313	+268
2146	11	+243	+178	+159	+318	+283	+215	+154	+221	+203	+198	+193	+413	+315	+393	+345

COLUMN NO. 4 B—(*Continued*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.		Longitudinal.												
		31 C	32 S	41 C	42 S	43 C	44 S	45 C	46 S	47 C	48 S	49 C	50 S	51 C	52 S	53 C
500	3	+004	+004	−025	+008	−020	−025	−018	−016	−021	−020	−025	−028	−028	−025	−019
1000	4	+012	+008	−066	−063	−047	−050	−041	−039	−053	−051	−055	−063	−050	−049	−044
1400	5	+024	+015	−104	−103	−076	−083	−066	−070	−089	−083	−085	−095	−079	−078	−066
1750	6	+079	+052	−223	−190	−147	−118	−093	−096	−144	−136	−149	−153	−129	−130	−085
1850	7	+108	+077	−303	−251	−212	−170	−121	−123	−189	−181	−211	−209	−176	−174	−109
1965	8	+169	+133	−398	−331	−300	−253	−145	−156	−230	−234	−298	−301	−240	−229	−139
2014	9	+204	+171	−465	−375	−370	−305	−166	−179	−260	−266	−349	−373	−289	−276	−155
2106	10	+277	+241	−535	−425	−462	−378	−238	−249	−336	−320	−450	−493	−369	−371	−178
2146	11	+333	+293	−570	−449	−522	−418	−295	−314	−344	−353	−535	−573	−425	−443	−186

Series 6: Scaling of vertical steel noticed at G. L. 42 S, 48 S, 70 S and 72 S.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.														
		54 S	55 C	56 S	57 C	58 S	59 C	60 S	61 C	62 S	63 C	64 S	65 C	66 S	67 C	68 S
500	3	−019	−021	−023	−029	−026	−028	Strain	−020	−025	−020	−025	−021	−021	−025	−021
1000	4	−045	−053	−054	−056	−053	−055	gage	−044	−050	−044	−060	−045	−048	−053	−048
1400	5	−068	−081	−085	−089	−085	−091	would	−071	−083	−069	−091	−071	−071	−083	−078
1750	6	−085	−129	−141	−141	−114	−140	not clear	−098	−126	−099	−135	−099	−103	−128	−119
1850	7	−119	−179	−198	−206	−154	−209	spirals	−136	−166	−133	−175	−131	−121	−180	−183
1965	8	−133	−254	−293	−303	−224	−323	and	−185	−204	−165	−235	−199	−170	−270	−264
2014	9	−141	−298	−351	−374	−276	−399	reach	−223	−235	−203	−290	−265	−229	−324	−320
2106	10	−154	−359	−439	−481	−371	−513	gage line	−256	−275	−245	−370	−355	−311	−421	−404
2146	11	−163	−390	−498	−600	−481	−514	on vertical steel	−259	−290	−280	−441	−409	−370	−484	−448

COLUMN NO. 4 B—(*Continued*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.												Lateral.		
		69 C	70 S	71 C	72 S	73 C	74 S	75 C	76 S	77 C	78 S	79 C	80 S	101 S	102 S	103 S
500	3	−023	−024	−028	−026	−019	−018	−026	−021	−025	−028	−024	−026	+004	+004	+005
1000	4	−050	−058	−060	−060	−040	−043	−059	−051	−053	−063	−056	−059	+009	+005	+009
1400	5	−078	−091	−091	−090	−059	−064	−095	−084	−085	−091	−088	−094	+019	+015	+014
1750	6	−132	−146	−145	−141	−083	−088	−209	−175	−149	−160	−155	−158	+045	+040	+021
1850	7	−183	−198	−205	−185	−104	−113	−308	−258	−219	−251	−203	−223	+041	+048	+014
1965	8	−250	−268	−275	−249	−161	−134	Not read	−400	−280	−409	−273	−316	+081	+086	+036
2014	9	−298	−326	−326	−323	−208	−154	−534	−493	−354	−504	−340	−388	+111	+119	+063
2106	10	−378	−430	−433	−441	−279	−180	Plug out	−654	−461		−460	−518	+154	+163	+109
2146	11	−441	−511	−513	−541	−332	−200			−548		−543		+175	+203	+135

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		104 S	105 S	106 S	107 S	108 S	109 S	110 S	111 S	112 S	113 S	114 S	115 S	116 S	117 S	118 S
500	3	+005	+003	+003	+001	+009	+005	+004	+001	+001	000	+003	+005	+006	+006	+002
1000	4	+008	+008	+006	+005	+010	+013	+010	+009	+009	+004	+005	+005	+013	+013	+005
1400	5	+015	+011	+006	+006	+020	+023	+011	+015	+013	+010	+014	+013	+021	+016	+011
1750	6	+040	+035	+024	+014	+028	+058	+040	+029	+035	+058	+028	+026	+041	+030	+024
1850	7	+035	+046	+024	+005	+016	+080	+059	+039	+050	+078	+054	+043	+064	+058	+038
1965	8	+069	+085	+055	+028	+045	+148	+110	+078	+101	+129	+095	+088	+111	+105	+080
2014	9	+094		+081	+051	+064	+184	+143	+105	+129	+168	+126	+111	+135	+145	+101
2106	10	+134		+136	+086	+101		+204	+134	+176		+185		+181	+215	+156
2146	11	+159	+208	+169	+109	+119	+360	+253	+153	+209	+374	+234	+154	+210	+261	+183

COLUMN NO. 4 B—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.													
		119 S	120 S	121 S	122 S	123 S	124 S	125 S	126 S	127 S	128 S	129 S	130 S	131 S	132 S
500	3	+008	+006	+009	+012	+003	+008	+005	+003	−001	+003	+003	+004	+007	+005
1000	4	+013	+013	+017	+020	+005	+012	+007	+008	+003	+008	+009	+005	+012	+012
1400	5	+022	+019	+019	+023	+015	+020	+007	+011	+011	+016	+019	+013	+023	+022
1750	6	+041	+047	+026	+028	+034	+040	+027	+030	+042	+054	+045	+042	+089	+075
1850	7	+061	+067	+038	+037	+051	+061	+053	+057	+069	+087	+078	+080	+133	+113
1965	8	+105	+117	+066	+068	+090	+106	+116	+121	+128	+165	+148	+158	+223	+199
2014	9	+134	+144	+088	+088	+118	+138	+149	+156	+165	+223	+190	+206	+308	+266
2106	10	+171	+202	+132	+120	+163	+194	+213	+226	+257	+348	+260	+290	+348	+423
2146	11	+195	+247	+151	+144	+199	+237	+254	+281	+344	+484	+314	+352		+553

COLUMN NO. 5.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		1 C	2 S	3 C	4 S	5 C	6 S	7 C	8 S	9 S	10 S	11 S	12 S	13 C	14 S	15 C
450	3	+005	+006	+004	+003	−003	+003	+007	+004	+005	+005	+003	+004	+008	+005	+001
900	4	+009	+016	+014	+013	+009	+003	+006	+013	+013	+012	+015	+012	+013	+007	+007
1100	5		+028		+023		+019		+023	+023	+022	+023	+027	+018	+010	+012
1350	6	+077	+068	+082	+057	+042	+048	+053	+060	+057	+051	+057	+054	+033	+025	+021
1500	7	+113	+101	+122	+091	+080	+073	+076	+083	+077	+073	+081	+078	+046	+038	+033
1700	8	+215	+197	+274	+197	+179	+155	+133	+154	+139	+136	+153	+159	+088	+081	+072
1900	9	+375	+333	+410	+310	+263	+228	+211	+252	+243	+234	+250	+233	+150	+144	+148
2060	10					+474	+312							+235	+223	

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		16 S	17 C	18 S	19 C	20 S	21 S	22 S	23 S	24 S	25 C	26 S	27 C	28 S	29 C	30 S
450	3	+006	+008	+011	+003	+003	+006	+004	+004	+017	+005	+006	+003	+004	+005	+005
900	4	+009	+013	+016	+008	+006	+005	+010	+010	+022	+014	+013	+008	+009	+009	+004
1100	5	+016	+021	+024	+012	+011	+009	+017				+020		+013		+015
1350	6	+026	+040	+039	+023	+024	+016	+019	+026	+031	+052	+044	+028	+045	+037	+036
1500	7	+038	+058	+058	+038	+039	+018	+025	+029	+035	+075	+064	+043	+044	+051	+056
1700	8	+073	+114	+119	+082	+087	+036	+045	+052	+063	+135	+112	+088	+087	+090	+098
1900	9	+152	+251	+233	+167	+174	+074	+073	+082	+099	+232	+193	+154	+142	+138	+154
2060	10		+381	+422							+404	+338			+208	+228

COLUMN NO. 5—(*Continued*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.		Longitudinal.											
		31 C	32 S	41 C	42 S	43 C	44 S	45 C	46 S	47 C	48 S	49 C	50 S	51 C	52 S
450	3	No hole	+004	−040	−034	−033	−034	−025	−029	−028	−034	−023	−036	−035	−031
900	4		+012	−083	−081	−078	−081	−070	−076	−073	−080	−070	−084	−079	−079
1100	5		+019	−111	−128	−118	−121	−103	−116	−108	−121	−120			
1350	6		+041	−183	−234	−215	−215	−185	−213	−180	−180	−221	−239	−175	−201
1500	7		+058	−228	−320	−303	−309	−240	−298	−243	−226	−295	−309	−223	−263
1700	8		+093	−378	−551	−535	−574	−394	−504	−371	−339	−460	−505	−321	−403
1900	9		+149	−620	−918	−780	−863	−525	−680	−556	−529	−650	−755	−543	−643
2060	10			−1354				−831							

Total Load, in Thousands of Pounds.	Series.	Longitudinal.													
		53 C	54 S	55 C	56 S	57 C	58 S	59 C	60 S	61 C	62 S	63 C	64 S	65 C	66 S
450	3	−028	−023	−036	−033	−024	−029	−026	−029	−034	−024	−025	−031	−030	−038
900	4	−071	−070	−083	−080	−058	−061	−061	−064	−078	−074	−064	−073	−056	−069
1100	5					−080	−088	−091	−092	−111	−100	−089	−100		
1350	6	−191	−164	−224	−201	−110	−129	−140	−139	−181	−128	−133	−175	−099	−114
1500	7	−264	−238	−304	−280	−141	−174	−189	−184	−253	−171	−181	−251	−124	−141
1700	8		−399	−503	−530	−184	−241	−280	−250	−443	−408	−319	−411	−173	−173
1900	9	−708	−675	−749	−791	−225	−325	−424	−358	−798	−608	−523	−655	−246	−233
2060	10					−1670	−1600			−1385	−1039				

COLUMN NO. 5—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.													
		67 C	68 S	69 C	70 S	71 C	72 S	73 C	74 S	75 C	76 S	77 C	78	79 C	80 S
450	3	−024	−026	−035	−031	−034	−030	−039	−036	−021	−030	−029	−031	−035	−035
900	4	−058	−056	−071	−071	−075	−066	−086	−084	−049	−070	−070	−075	−093	−089
1100	5							−116	−111	−088	−095	−095	−101	−129	−126
1350	6	−101	−099	−138	−131	−133	−121	−193	−179	−129	−158	−134	−139	−194	−213
1500	7	−130	−123	−181	−175	−174	−168	−249	−239	−159	−211	−168	−178	−241	−261
1700	8	−180	−158	−283	−265	−265	−238	−390	−378	−260	−341	−236	−251	−331	−350
1900	9	−251	−214	−434	−385	−418	−463	−630	−611	−449	−529	−568	−364	−470	−498
2060	10							−733	−903			−1820	−1910		

COLUMN NO. 5 A.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		1 C	2 S	3 C	4 S	5 C	6 S	7 C	8 S	9 S	10 S	11 S	12 S	13 C	14 S	15 C
400	3	+004	+004	+002	000	+005	+007	+002	+005	+003	+003	+007	+003	+003	+003	+003
800	4	+014	+011	+010	+008	+015	+010	+008	+008	+009	+009	+015	+008	+013	+010	+008
1000	5	+022	+018	+016	+013	+024	+020	+013	+013	+013	+017	+018	+011	+015	+014	+015
1200	6	+040	+030	+035	+026	+033	+026	+022	+018	+019	+016	+017	+016	+022	+021	+019
1350	7	+067	+047	+060	+043	+051	+037	+033	+031	+029	+023	+020	+022	+032	+030	+026
1500	8	+099	+071	+087	+064	+076	+061	+051	+044	+044	+034	+023	+030	+048	+044	+039
1700	9	+158	+117	+130	+102	+127	+098	+095	+081	+076	+061	+067	+060	+090	+081	+076
1850	10	+220	+171	+211	+165	+206	+152	+134	+122	+113	+088	+091	+096	+133	+120	+106
2233	11	+505	+424	+625	+484	Plug loose	+489	+359	+371	+398	+307	+289	+1016*	+598	+545	+289

* Seems doubtful.

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		16 S	17 C	18 S	19 C	20 S	21 S	22 S	23 S	24 S	25 C	26 S	27 C	28 S	29 C	30 S
400	3	+005	+004	+006	+005	+007	+008	+013	+004	+004	+003	+005	+002	−003	+003	+003
800	4	+011	+014	+015	+013	+013	+014	+018	+010	+012	+005	+009	+005	+003	+011	+005
1000	5	+015	+019	+019	+018	+018	+019	+022	+012	+015	+014	+014	+009	+008	+015	+008
1200	6	+019	+023	+023	+023	+024	+023	+019	+017	+018	+018	+018	+016	+012	+029	+020
1350	7	+026	+034	+031	+035	+035	+036	+031	+026	+026	+030	+028	+023	+019	+043	+028
1500	8	+037	+061	+043	+050	+047	+044	+038	+033	+036	+043	+038	+033	+030	+063	+048
1700	9	+073	+100	+082	+093	+083	+080	+062	+064	+067	+088	+075	+077	+076	+121	+100
1850	10	+104	+154	+114	+133	+112	+118	+096	+095	+098	+132	+113	+120	+119	+178	+147
2233	11	+299	+429	+319	+450	+410	+403	+243	+318	+460	+626	+550				

COLUMN No. 5 A—(*Continued*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.		Longitudinal.											
		31 C	32 S	41 C	42 S	43 C	44 S	45 C	46 S	47 C	48 S	49 C	50 S	51 C	52 S
400	3	+003	+002	−024	−025	−024	−023	−023	−028	−020	−028	−029	−024	−019	−020
800	4	+012	+013	−064	−066	−064	−050	−055	−064	−051	−054	−064	−055	−053	−053
1000	5	+015	+013	−085	−089	−088	−078	−074	−081	−068	−069	−088	−079	−076	−074
1200	6	+025	+023	−121	−118	−123	−144	−101	−109	−085	−088	−113	−103	−100	−100
1350	7	+043	+036	−169	−165	−168	−214	−139	−163	−108	−115	−146	−136	−131	−123
1500	8	+059	+047	−226	−221	−214	−280	−181	−214	−128	−140	−184	−174	−166	−131
1700	9	+096	+086	−344	−334	−323	−421	−273	−313	−171	−180	−271	−264	−238	−181
1850	10	+129	+119	−458	−443	−440	−585	−394	−430	−223	−239	−369	−369	−313	−246
2233	11	+618	+563	−856	−891	−889		−1056	−880	−563	−625	−995	−1045	−848	−713

Total Load, in Thousands of Pounds.	Series.	Longitudinal.													
		53 C	54 S	55 C	56 S	57 C	58 S	59 C	60 S	61 C	62 S	63 C	64 S	65 C	66S
400	3	−023	−023	−025	−026	−021	−025	−033	−030	−028	−034	−028	−028	−025	−024
800	4	−054	−054	−050	−056	−045	−055	−061	−058	−054	−060	−059	−054	−054	−052
1000	5	−076	−069	−064	−066	−064	−075	−076	−073	−070	−083	−075	−075	−071	−070
1200	6	−104	−091	−081	−090	−084	−099	−100	−095	−094	−104	−095	−084	−096	−095
1350	7	−133	−111	−101	−130	−108	−121	−120	−118	−118	−131	−126	−126	−121	−116
1500	8	−164	−136	−123	−169	−143	−143	−151	−146	−156	−178	−160	−170	−150	−150
1700	9	−241	−214	−174	−253	−214	−198	−216	−193	−233	−271	−243	−271	−223	−216
1850	10	−321	−299	−245	−346	−295	−276	−278	−254	−319	−355	−339	−381	−300	−281
2233	11	−668	−638	−505	−739	−1096	−1114	−660	−625	−646	−704	−1016	−1084	−878	−824

COLUMN No. 5 A—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.													
		67 C	68 S	69 C	70 S	71 C	72 S	73 C	74 S	75 C	76 S	77 C	78 S	79 C	80 S
400	3	−025	−028	−024	−025	−028	−029	−024	−019	−026	−015	−025	−025	−025	−024
800	4	−054	−058	−050	−051	−060	−060	−054	−053	−056	−046	−060	−060	−061	−065
1000	5	−070	−070	−064	−066	−079	−080	−074	−075	−070	−060	−075	−078	−084	−085
1200	6	−090	−094	−086	−089	−106	−106	−099	−098	−093	−088	−105	−108	−118	−118
1350	7	−109	−118	−108	−116	−136	−131	−125	−131	−121	−119	−145	−146	−159	−155
1500	8	−134	−145	−135	−145	−178	−170	−160	−176	−158	−166	−190	−199	−199	−195
1700	9	−173	−200	−189	−196	−265	−263	−231	−254	−248	−270	−299	−326	−296	−285
1850	10	−224	−264	−239	−258	−363	−368	−311	−333	−354	−386	−423	−460	−394	−378
2233	11	−429	−576	−674	−790			−875	−875						−1115

COLUMN NO. 6.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.													
		1 C	3 C	4 S	5 C	7 C	9a C	10 S	10a C	11a C	12a C	13 C	17 C	19 C	21a C	22a C
700	3	+004	+008	+012	+006	+007	+006	+004	+009	+006	+004	+006	−008	+007	+006	+021
1150	4	+011	+025	+028	+010	+016	+012	+012	+023	+012	+013	+013	+004	+009	+011	+027
1350	5	+021	+057	+044	+015	+022	+013	+018	+038	+016	+018	+013	+008	+017	+018	Holes
1550	6	+068	+077	+073	+030	+043	+023	+037	+064	+024	+029	+019	+020	+029	+023	in gage
1740	7	+344	+161	+162	+117	−077	Plug	+097	+171	+093	+082	+027	+048	+038	+032	line not
1648	8		+320	+313			loose	+323	+472	+178	+329	+038	+085	+053	+027	drilled true

Series 7: G. L. 7 C, reading seems questionable.

Total Load, in Thousands of Pounds.	Series.	Lateral.						Longitudinal.							
		23a C	24a C	25 C	27 C	29 C	31 C	41 C	43 C	44 S	45 C	47 C	49 C	51 C	52 S
700	3	+001	+007	+003	+010	+013	−005	−030	−039	−043	−038	−029	−030	−038	−040
1150	4	+019	+014	−002	+014	+028	+003	−078	−093	−088	−060	−049	−074	−080	−073
1350	5	+027	+018	+002	+018	+041	+015	−103	−128	−108	−073	−064	−095	−100	−098
1550	6	+038	+027	+009	+022	+068	+023	−153	−205	−158	−105	−084	−130	−129	−120
1740	7	+065	+046	+014	+024	+113	+038	−160	−378	−285	−156	−129	−195	−186	−169
1648	8	+096	+056	+007	+025	+173	−043	Plug loose					−109		

Series 8: Concrete crushed at gage lines 45 C., 47 C.

COLUMN NO. 6—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.													
		53 C	55 C	57 C	59 C	61 C	63 C	65 C	67 C	69 C	71 C	73 C	75 C	77 C	79 C
700	3	−041	−034	−021	Holes	−044	−026	−019	−021	−054	−030	−011	−041	−059	−031
1150	4	−065	−063	−048	for	−069	−050	−045	−056	−066	−058	−034	−066	−100	−058
1350	5	−079	−083	−060	gage	−089	−061	−060	−069	−096	−076	−041	−080	−126	−080
1550	6	−109	−110	−073	line	−116	−079	−073	−084	−121	−096	−051	−093	−198	−101
1740	7	−185	−170	−085	not	−149	−105	−084	−099	−150	−133	−059	−104	−245	−134
1648	8	−188	−173	−088	drilled	−189	−109	−063	−100	−198	−143	−040	−106	−278	−153

COLUMN NO. 6 A.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.													
		1 C	3 C	5 C	7 C	9a C	10a C	11a C	12a C	13 C	15 C	17 C	19 C	21a C	22a C
400	3	−002	+002	+004	+003	+008	+003	+001	+003	+005	+003	+012	+009	+005	+006
800	4	+005	+009	+008	+008	+010	+008	+002	+003	+005	+008	+011	+014	+008	+014
1000	5	+012	+011	+011	+010	+016	+017	+006	+008	+010	+010	+013	+016	+018	+016
1200	6	+016	+011	+010	+011	+021	+018	+005	+012	+023	+018	+020	+025	+024	+020
1400	7	+021	+017	+012	+014	+032	+033	+008	+021	+028	+012	+017	+029	+052	+035
1600	8	+027	+020	+015	+018	+043	+053	+011	+020	+048	+028	+033	+048	+117	+062
1720	9	+037	+030	+025	+023	+061	+087	+022	+024	+068	+038	+049	+068	+197	+139

Total Load, in Thousands of Pounds.	Series.	Lateral.						Longitudinal.						
		23a C	24a C	25 C	27 C	29 C	31 C	41 C	43 C	45 C	47 C	49 C	51 C	53 C
400	3	−001	+004	−003	+002	−002	+003	−025	−015	−008	−020	−021	−020	−005
800	4	000	+017	+008	+006	−001	+006	−055	−031	−025	−020	−046	−048	−024
1000	5	−001	+024	+017	+009	+004	+013	−069	−043	−035	−058	−063	−085	−029
1200	6	+003	+042	+026	+018	+008	+021	−086	−051	−045	−073	−081	−104	−044
1400	7	+005	+080	+052	+050	+014	+037	−115	−073	−068	−090	−100	−123	−059
1600	8	+043	+203	+134	+153	+062	+093	−131	−085	−086	−104	−123	−160	−079
1720	9	+158				+150	+196	−148	−099	−111	−116	−139	−203	−104

COLUMN NO. 6 A—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.												
		55 C	57 C	59 C	61 C	63 C	65 C	67 C	69 C	71 C	73 C	75 C	77 C	79 C
400	3	+005	−035	−021	−004	−019	−033	−015	−015	−020	−030	−021	−008	−024
800	4	−039	−061	−038	−020	−040	−071	−033	−034	−049	−060	−045	−030	−049
1000	5	−053	−078	−050	−026	−051	−090	−044	−045	−065	−079	−060	−043	−066
1200	6	−068	−100	−066	−043	−070	−114	−056	−061	−090	−105	−079	−056	−088
1400	7	−083	−119	−085	−058	−086	−130	−074	−086	−133	−139	−115	−083	−118
1600	8	−099	−143	−111	−085	−106	−225	−100	−141	−239	−194	−181	−140	−153
1720	9	−106	−154	−128	−120	−125	−274	−121	−283	−459	+108	−366	−203	−205

Reading on G. L. 73 C for Series 9 is affected by adjacent crack.

COLUMN NO. 7.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.													
		1 C	3 C	5 C	7 C	9a C	10a C	11a C	12a C	13 C	15 C	17 C	19 C	21a C	22a C
250	3	+004	+004	+003	+003	+003	+004	−006	+007	000	+003	+002	+001	−002	+011
500	4	+009	+006	+006	+004	+005	+011	+007	+016	+008	+007	+005	+008	+003	+015
750	5	+014	+009	+008	+008	+010	+013	+012	+018	+014	+016	+013	+011	+007	+018
850	6	+020	+013	+008	+008	+013	+017	+015	+023	+018	+016	+014	+014	+010	+022
950	7	+025	+013	+010	+013	+015	+022	+019	+023	+020	+025	+018	+023	+014	+027
1000	8	+029	+017	+011	+017	+018	+022	+021	+027	+029	+028	+020	+028	+016	+029

Total Load, in Thousands of Pounds.	Series.	Lateral.						Longitudinal.						
		23a C	24a C	25 C	27 C	29 C	31 C	41 C	43 C	45 C	47 C	49 C	51 C	53 C
250	3	+006	−010	Holes	+003	+003	+001	−011	−009	−015	−010	−013	−024	−014
500	4	+013	−001	in gage	+011	+008	+005	−034	−025	−030	−025	−033	−041	−031
750	5	+026	+010	line	+028	+019	+008	−066	−045	−046	−049	−059	−061	−051
850	6	+037	+016	not	+044	+027	+014	−075	−055	−055	−059	−071	−073	−058
950	7	+039	+020	drilled	+088	+058	+022	−089	−065	−059	−071	−085	−083	−066
1000	8	+048	+028	true	+243	+152	+053	−101	−070	−063	−080	−098	−085	−069

COLUMN NO. 7—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.												
		55 C	57 C	59 C	61 C	63 C	65 C	67 C	69 C	71 C	73 C	75 C	77 C	79 C
250	3	−011	−025	−016	−016	−008	−004	−016	−020	−008	Rope	−020	−021	−014
500	4	−033	−050	−040	−038	−029	−023	−025	−045	−029	in	−044	−045	−035
750	5	−058	−076	−060	−056	−054	−041	−060	−074	−055	way	−074	−075	−055
850	6	−071	−089	−070	−066	−064	−053	−070	−090	−070		−081	−091	−068
950	7	−085	−104	−079	−076	−079	−064	−079	−104	−086		−100	−108	−079
1000	8	−095	−124	−088	−084	−094	−079	−089	−118	−106		−126	−111	−089

Gage line 77 C was read with strain gage inverted to avoid interference with safety rope tied around upper end of column shaft.

COLUMN NO. 7 A.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.												
		1 C	3 C	5 C	7 C	9a C	10a C	11a C	12a C	13 C	15 C	17 C	19 C	21a C	22a C
250	3	+005	+001	+005	+003	+004	+009	+005	+003	+006	+011	+005	+004	+004	+003
500	4	+011	+005	+010	+010	+007	+013	+009	+007	+008	+014	+007	+008	+006	+008
750	5	+018	+005	+012	+013	+009	+022	+018	+008	+009	+023	+014	+010	+018	+013
860	6	+014	+009	+018	+014	+008	+026	+021	+009	+010	+027	+021	+010	+014	+019

Total Load, in Thousands of Pounds.	Series.	Lateral.						Longitudinal.						
		23a C	24a C	25 C	27 C	29 C	31 C	41 C	43 C	45 C	47 C	49 C	51 C	53 C
250	3	+003	+006	+003	+005	+002	+005	−018	−020	−014	−010	−013	−013	−018
500	4	+004	+009	+009	+007	+003	+007	−035	−041	−034	−028	−036	−033	−036
750	5	+008	+015	+071	+019	+005	+026	−056	−071	−061	−043	−055	−056	−064
860	6	+014	+017			−002	+378	−036	−073	−070	−039	−033	−060	−081

Total Load, in Thousands of Pounds.	Series.	Longitudinal.												
		55 C	57 C	59 C	61 C	63 C	65 C	67 C	69 C	71 C	73 C	75 C	77 C	79 C
250	3	−013	−019	−014	−004	−015	−019	−009	−010	−019	−026	−009	−006	−013
500	4	−029	−041	−034	−020	−036	−044	−038	−029	−039	−056	−030	−016	−040
750	5	−048	−065	−055	−040	−056	−068	−066	−048	−053	−103	−056	−025	−079
860	6	−044	−041	−058	−066	−058	−090	−086	−080	−080	+034	−141	+020	−185

COLUMN NO. 8.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.													
		1 C	3 C	5 C	7 C	9a C	10a C	11a C	12a C	13 C	15 C	17 C	19 C	21a C	22a C	23a C
400	3	+002	+006	000	+003	000	+003	+002	+014	+003	+004	+002	+001	−002	+003	+002
800	4	+005	+008	+008	+010	+006	+018	+015	+020	+013	+015	+005	+018	+012	+013	+008
1040	5	−004	−002	+005	+006	+300						+012		+019	+003	−008

Series 5. Deformations of little value. Load fell suddenly from maximum to about 200,000 lbs., throwing many gage lines beyond range of instruments. Read after column showed failure at middle.

Total Load, in Thousands of Pounds.	Series.	Lateral.					Longitudinal.									
		24a C	25 C	27 C	29 C	31 C	41 C	42 S	43 C	44 S	45 C	46 S	47 C	48 S	49 C	50 S
400	3	−001	+003	−003	−003	−001	−023	−020	−016	−021	−032	−023	−021	−027	−017	−021
800	4	+003	+038	+008	+008	+003	−053	−046	−040	−048	−071	−064	−048	−064	−049	−057
1040	5	−003	+078	+023	+004	−002	−026	−036	−019	−035	−018	−071	−025	−062	−085	−082

COLUMN No. 8—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.														
		51 C	52 S	53 C	54 S	55 C	56 S	57 C	58 S	59 C	60 S	61 C	62 S	63 C	64 S	65 C
400	3	−019	−018	−019	−026	−020	−020	−026	−033	−028	−028	−021	−024	−023	−024	−031
800	4	−059	−063	−065	−054	−051	−058	−064	−071	−069	−068	−062	−067	−066	−067	−072
1040	5	−344				−002	−123	−079	−236	−113	−127	−033	−043	−057	−101	−046

Total Load, in Thousands of Pounds.	Series.	Longitudinal.														
		66 S	67 C	68 S	69 C	70 S	71 C	72 S	73 C	74 S	75 C	76 S	77 C	78 S	79 C	80 S
400	3	−030	−026	−029	−025	−023	−034	−027	−035	−038	−026	−026	−026	−028	−025	−021
800	4	−072	−070	−070	−057	−061	−073	−064	−090	−088	−078	−076	−073	−058	−066	−062
1040	5	−043	−037	−041	−030	−033	−038	−032	−043	−029	−053	−048	−053	−039	−034	−028

COLUMN NO. 8 A.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.												
		1 C	3 C	5 C	7 C	9a C	10a C	11a C	12a C	13 C	15 C	17 C	19 C	21a C	22a C
300	3	+004	+008	+008	+003	+001	+004	+003	+005	+005	+003	+002	+004	000	+002
600	4	+007	+012	+013	+010	+003	+011	+011	+009	+008	+003	+008	+012	+003	+006
800	5	+008	+017	+013	+010	+006	+015	+018	+011	+013	+007	+015	+017	+008	+008
1000	6	+010	+021	+023	+018	+009	+024	+024	+019	+017	+015	+029	+023	+012	+014
1200	7	+017	+029	+022	+026	+014	+035	+041	+037	+023	+028	+064	+046	+017	+023
1360	8	+019	+023	+013	+013		+057	+053							

Total Load, in Thousands of Pounds.	Series.	Lateral.						Longitudinal.						
		23a C	24a C	25 C	27 C	29 C	31 C	41 C	43 C	45 C	47 C	49 C	51 C	53 C
300	3	+005	+007	+001	+002	+003	+005	−011	−014	−016	−013	−011	−013	−019
600	4	+007	+005	+003	+003	+008	+012	−034	−033	−039	−035	−034	−036	−046
800	5	+011	+018	+006	+007	+012	+018	−053	−049	−055	−051	−049	−051	−064
1000	6	+017	+018	+008	+014	+019	+022	−074	−061	−069	−070	−066	−074	−085
1200	7	+023	+022	+013	+023	+034	+037	−101	−076	−081	−088	−093	−103	−111
1360	8	+004	+018	+006	+008	+027	+039	−059	−034	−001	−015		−103	−083

Series 8: Large cracks near gage lines 49 and 59.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.												
		55 C	57 C	59 C	61 C	63 C	65 C	67 C	69 C	71 C	73 C	75 C	77 C	79 C
300	3	−011	−010	−015	−019	−019	−004	−013	−016	−011	000	−009	−018	−003
600	4	−034	−031	−038	−046	−046	−025	−034	−039	−038	−018	−028	−045	−031
800	5	−051	−048	−055	−065	−066	−040	−059	−059	−059	−030	−044	−065	−046
1000	6	−070	−059	−071	−086	−084	−061	−079	−079	−073	−038	−065	−088	−064
1200	7	−096	−078	−100	−128	−119	−083	−106	−096	−091	−056	−083	−125	−084
1360	8	−125	−044		−090				−021	−041	−024	−041	−039	−043

Series 8: Large cracks near gage lines 61, 63, 65, 67.. Read after column showed failure at middle.

COLUMN NO. 9.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.														
		1 C	3 C	5 C	7 C	9a C	10a C	11a C	12a C	13 C	15 C	17 C	19 C	21a C	22a C	23a C
400	3	+003	+005	+002	+002	+003	+001	+002	+002	+003	−008	000	+005	+005	+003	+002
800	4	+008	+010	+006	+005	+013	+008	+007	+006	+008	000	+004	+010	+015	+011	+003
1000	5	+012	+016	+009	+008	+016	+014	+011	+008	+008	+003	+006	+013	+031	+023	+007
1150	6	+010	+015	+013	+008	+013	+007	+013	+008	+011	+082	+138	+196			

Total Load, in Thousands of Pounds.	Series.	Lateral.					Longitudinal.									
		24a C	25 C	27 C	29 C	31 C	41 C	42 S	43 C	44 S	45 C	6 S	47 C	48 S	49 C	50 S
400	3	+003	+008	−001	−001	+002	−019	−021	−019	−020	−018	−017	−009	−020	−021	−024
800	4	+009	+024	+011	+003	+012	−043	−047	−055	−052	−053	−060	−040	−049	−054	−059
1000	5	+013	+066	+031	+003	+008	−059	−065	−072	−068	−068	−072	−059	−066	−072	−073
1150	6			+254	+082	+384	−034	−033	−043	−040	−053	−055	−038	−051	−044	−043

COLUMN NO. 9—(*Concluded*).

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.														
		51 C	52 S	53 C	54 S	55 C	56 S	57 C	58 S	59 C	60 S	61 C	62 S	63 C	64 S	65 C
400	3	−023	−018	−022	−022	+004	−020	−018	−028	−024	−031	−026	−022	−012	−024	+004
800	4	−053	−059	−061	−058	−020	−053	−049	−058	−052	−054	−054	−052	−035	−059	−024
1000	5	−073	−074	−078	−075	−037	−067	−066	−075	−065	−078	−062	−080	−048	−073	−046
1150	6	−040	−038	−059	−062	−025	−053	−028	−053	−032	−090	−048	−111	+006	−088	

Total Load, in Thousands of Pounds.	Series.	Longitudinal.														
		66 S	67 C	68 S	69 C	70 S	71 C	72 S	73 C	74 S	75 C	76 S	77 C	78 S	79 C	80 S
400	3	−018	−020	−020	−015	−013	−023	−023	Holes	−024	−028	−018	−020	−023	−023	−027
800	4	−063	−061	−060	−055	−048	−057	−058	for gage	−070	−072	−060	−054	−056	−055	−052
1000	5	−086	−087	−089	−076	−069	−076	−078	line not	−090	−122	−104	−077	−074	−078	−076
1150	6	−253		−299		−466	−452	−358	drilled true	−145	−208	−160	−058	−107	−128	−196

COLUMN NO. 9 A.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.												
		1 C	3 C	5 C	7 C	9a C	10a C	11a C	12a C	13 C	15 C	17 C	19 C	21a C	22a C
300	3	000	+001	+001	+001	+003	+003	+003	+007	+001	+004	+008	+007	−001	+004
600	4	+004	+007	+003	+004	+006	+008	+011	+012	+006	+008	+017	+012	+013	+008
800	5	+006	+008	+007	+008	+005	+011	+012	+019	+011	+010	+019	+016	+017	+011
950	6	+011	+012	+013	+015	+012	+013	+017	+022	+018	+016	+025	+024	+023	+015
1100	7	+020	+022	+023	+020	+020	+024	+025	+033	+024	+020	+032	+033	+033	+019
1206	8	+021	+023	+026	+019	+023	+028	+030	+035	+033	+036	+046	+035	+128	+077

Series 8: Large cracks near gage line 21 aC affects reading.

Total Load, in Thousands of Pounds.	Series.	Lateral.						Longitudinal.						
		23a C	24a C	25 C	27 C	29 C	31 C	41 C	43 C	45 C	47 C	49 C	51 C	53 C
300	3	−006	+003	+005	+002	+009	+006	−014	−013	−011	−015	−023	−011	−019
600	4	−001	+011	+010	+003	+013	+011	−039	−038	−039	−038	−045	−035	−041
800	5	+005	+013	+013	+003	+018	+022	−054	−054	−049	−053	−063	−050	−056
950	6	+013	+025	+020	+008	+027	+041	−066	−069	−063	−065	−078	−064	−071
1100	7	+019	+043	+039	+010	+038	+088	−084	−091	−078	−076	−091	−083	−088
1206	8	+105	+126	+270	+028	+257		−086	−094	−086	−078	−095	−101	−126

Series 8: Large cracks near gage lines 21a C, 23a C, 24a C, 25 C, 29 C, and 31 C affect readings.

Total Load, in Thousands of Pounds.	Series.	Longitudinal.												
		55 C	57 C	59 C	61 C	63 C	65 C	67 C	69 C	71 C	73 C	75 C	77 C	79 C
300	3	−020	−023	−013	−014	−014	−025	−015	−011	−019	−010	−009	−013	−024
600	4	−048	−045	−035	−034	−039	−051	−034	−030	−041	−033	−029	−031	−051
800	5	−065	−064	−046	−050	−055	−073	−046	−039	−063	−053	−038	−048	−076
950	6	−084	−081	−060	−064	−074	−091	−059	−056	−080	−073	−046	−059	−098
1100	7	−095	−098	−079	−084	−090	−121	−085	−079	−110	−098	−058	−086	−128
1206	8	−096	−149	−106	−111	−101	−164	−145	−120	−153	−101	−064	−111	−248

COLUMN NO. 10.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.													
		1 C	3 C	5 C	7 C	9a C	10a C	11a C	12a C	13 C	15 C	17 C	19 C	21a C	22a C
300	3	+001	+001	000	000	000	+002	000	+003	000	+001	+003	000	+003	000
600	4	+005	000	+002	−001	+003	−002	−004	+011	+001	+003	+003	+002	+007	+001
800	5	+003	000	+002	+004	+004	+001	−001	+010	+006	+002	+005	+003	+006	+003
1000	6	+010	+003	+004	+009	+008	+005	000	+019	+011	+004	+004	+008	+008	+005
1200	7	+017	+008	+003	+015	+016	+011	+007	+022	+017	+008	+010	+013	+013	+007
1451	8	+047	+057	+024	+018	+046	+026	+019	+036	+061	+028	+033	+026	+075	+051

Total Load, in Thousands of Pounds.	Series.	Lateral.						Longitudinal.						
		23a C	24a C	25 C	27 C	29 C	31 C	41 C	43 C	45 C	47 C	49 C	51 C	53 C
300	3	−002	+003	+002	Holes	000	+003	−020	−014	−010	−016	−018	−011	−011
600	4	+003	+002	+003	for	+003	+004	−039	−031	−025	−036	−040	−033	−048
800	5	+003	+001	+005	gage	+006	+008	−055	−046	−038	−049	−054	−045	−048
1000	6	+013	+007	+007	line not	+010	+015	−075	−064	−051	−069	−068	−063	−070
1200	7	+009	+013	+015	drilled	+023	+033	−095	−084	−070	−093	−086	−080	−085
1451	8	+027	+049	+273	true	+223		−153	−139	−111	−129	−126	−120	−109

Total Load, in Thousands of Pounds.	Series.	Longitudinal.												
		55 C	57 C	59 C	61 C	63 C	65 C	67 C	69 C	71 C	73 C	75 C	77 C	79 C
300	3	−019	−020	−018	−006	−015	−014	−015	−014	−010	−009	−014	−015	−016
600	4	−031	−039	−038	−030	−036	−033	−033	−035	−030	−036	−031	−041	−040
800	5	−050	−049	−048	−043	−049	−043	−041	−050	−050	−049	−044	−055	−058
1000	6	−064	−066	−064	−060	−069	−059	−059	−066	−071	−065	−058	−076	−085
1200	7	−089	−081	−080	−075	−085	−078	−078	−085	−088	−084	−075	−103	−111
1451	8	−120	−124	−119	−100	−119	−171	−174	−151	−159	−209	−203	−296	−304

COLUMN No. 10 A.

UNIT-DEFORMATIONS IN HUNDRED-THOUSANDTHS OF AN INCH.

Total Load, in Thousands of Pounds.	Series.	Lateral.													
		1 C	3 C	5 C	7 C	9a C	10a C	11a C	12a C	13 C	15 C	17 C	19 C	21a C	22a C
300	4	+008	+004	+005	+002	000	+003	+001	+003	+004	+003	+003	+007	+006	+002
600	5	+017	+012	+010	+005	+008	+007	+006	+008	+010	+008	+008	+012	+004	+009
800	6	+018	+017	+012	+008	+013	+011	+008	+011	+013	+011	+010	+015	+010	+011
1000	7	+023	+023	+015	+009	+018	+014	+012	+016	+016	+013	+013	+020	+008	+014
1200	8	+037	+043	+021	+015	+028	+019	+021	+026	+020	+017	+021	+023	+012	+018
1400	9	+096	+155	+038	+029	+075	+033	+041	+071	+025	+024	+036	+033	+013	+028

Total Load, in Thousands of Pounds.	Series.	Lateral.						Longitudinal.						
		23a C	24a C	25 C	27 C	29 C	31 C	41 C	43 C	45 C	47 C	49 C	51 C	53 C
300	4	+004	+002	+001	+004	000	+008	−019	−013	−011	−011	−016	−013	−013
600	5	+007	+008	+004	+013	+005	+008	−046	−041	−028	−030	−043	−034	−031
800	6	+008	+011	+005	+014	+008	+013	−060	−056	−039	−038	−059	−048	−044
1000	7	+013	+015	+017	+018	+015	+017	−084	−079	−058	−054	−074	−063	−061
1200	8	+019	+018	+021	+037	+026	+023	−121	−115	−093	−079	−096	−078	−084
1400	9	+026	+026	+028	+059	+062	+045	−196	−213	−144	−109	−148	−089	−109

Total Load, in Thousands of Pounds.	Series.	Longitudinal.												
		55 C	57 C	59 C	61 C	63 C	65 C	67 C	69 C	71 C	73 C	75 C	77 C	79 C
300	4	−016	−018	−024	−018	−023	−014	−018	−019	−018	−013	−013	−016	−020
600	5	−035	−033	−041	−035	−043	−033	−038	−039	−035	−031	−033	−038	−040
800	6	−049	−045	−056	−051	−054	−043	−050	−053	−046	−040	−044	−050	−055
1000	7	−065	−058	−073	−069	−069	−055	−066	−069	−060	−053	−060	−068	−069
1200	8	−089	−070	−086	−090	−089	−073	−085	−091	−076	−068	−083	−090	−093
1400	9	−138	−084	−103	−108	−110	−088	−109	−118	−095	−089	−111	−125	−133

DISCUSSION.

MR. WILLIS WHITED.—How thick was the layer of plaster of Paris? **Mr. Whited.**

PROF. A. N. TALBOT.—Just as thin as it could be made, less than $\frac{1}{16}$ of an inch. **Mr. Talbot.**

MR. EDWARD GODFREY.—It would be a serious mistake to use the data of these tests for the designing of columns, for three reasons, namely (1) because these were not columns, (2) because of the way they were made, (3) because of the way they were tested. **Mr. Godfrey.**

The cylindrical portion of these shafts was five or six diameters in length. The Joint Committee Report defines this as massive concrete. It would be just as serious a mistake to use the data of these tests in designing columns as it would be to use tests on short cast iron cylinders for the design of cast iron columns. Cast iron cylinders will stand probably 100,000 lbs. per sq. in. Cast iron columns are designed for loads of 7,000 to 10,000 lbs. per sq. in. This is 7 to 10 per cent of the ultimate compressive strength of the metal.

These tests have scarcely any value as showing what a column will carry in a real building and only a restricted value in showing what piers or massive concrete will carry, and this is because of the way they were made. I have personally watched the building of piers, not of concrete but of brick, in the laboratory where these were made. Each brick was carefully dipped in water. Every possible crevice was flushed with cement mortar. This parlor method of laying brick would never be realized in a building unless an inspector were standing over each bricklayer. Doubtless corresponding methods were used in making these massive concrete piers. The conditions in an actual building are not even approached in a laboratory test. Every effort possible is made to make them different. In a building the concrete is roughly made, mixed for a fraction of a minute, dropped down a long box, where it is impossible to see if it enters all corners, poked a little with a long pole, plastered up if any cavities show up when the forms are removed. It is impossible to see if the rods are in

Mr. Godfrey. just the proper place. Then this column and the entire monolith is subject to shrinkage, the loads are eccentric, the deflections of beams add to eccentric stresses, and the builder confidently claims that he has a factor of safety of four or five based on the showing of these nursery piers. One such actual building column that I observed failed at 200 lbs. per sq. in. when it was jarred by a failure in another part of the building. A short cylinder cut out of this, or one made with great heads, as these were, in a laboratory would doubtless have stood loads about the same as these.

It is false interpretation of tests to take the results of these pier tests, with the infinite care used in their preparation and testing, and apply them to the design of buildings.

I do not advocate the making of slip-shod tests, but I do advocate care in applying the result of ideal tests to rough practical structures.

There are two ways of discovering experimentally the value of columns as load carriers. One is by laboratory tests, the other is by wrecked buildings. I say wrecked buildings because these have furnished the most enlightening facts. Buildings that have not collapsed do not prove much of anything, for it is rare that the columns of a building are loaded even to their safe load. A large amount of load would be required to load even a few columns of a building to only their safe load, hence the sinister showing of collapsed reinforced concrete buildings, where almost invariably the rodded columns break up into chunks or crumble, not even standing alone with no burden to carry. But unfortunately the clear showing of these wrecks is perverted by zealous but mistaken defenders of concrete, and this leaves the laboratory test as a guide. It is an unfortunate fact that these tests do not represent actual conditions in a structure. It is toughness that a column needs, and it is toughness that the rodded column, the prevailing style, lacks entirely. Furthermore it is the toughness, or the lack of it, of a column that the average laboratory test fails to discover because of the test and of its preparation.

If a long column were tested, something of the quality of toughness could be determined, or if a pair of columns and the supported beam were tested, designed as standard rules allow.

While these tests show little of the action of a column, they do show the absolutely erratic results that generally accompany

the use of longitudinal steel, even in the parlor test. Take for example the first three groups of hooped columns. Those having 1 per cent of longitudinal steel are $1\frac{1}{2}$ per cent stronger on the average, instead of 15 per cent as per standard rules. Standard rules just 900 per cent off. One specimen with no longitudinal steel is stronger than any of those with 1 per cent longitudinal steel and only exceeded by one of the group with 2 per cent longitudinal steel. What is the standard rule worth? What do averages signify in erratic tests of this sort? Why did not the Committee give as one of the "conclusions" that longitudinal steel has no definite effect one way or the other on a hooped column so far as supporting vertical loads are concerned? Because this would be a bitter truth and disturb all the standards of the profession. Mr. Godfrey.

Look at Group 4 with 6 per cent longitudinal steel! One specimen is only 10 per cent stronger than test 1*b*.

Please observe I am not advocating the omission of longitudinal steel rods in a column. They are needed for toughness but useless for compression.

Look at the rodded columns! Group 9 with 2 per cent longitudinal steel is weaker than Group 8 with 1 per cent. Someone will say, "Erratic test 8*a*." Erratic tests in rodded columns are the rule and not the exception.

In a building the upright rods are not perfectly straight, they are not milled on the ends and brought to a perfect bearing, the load is not applied through a greased spherical head and in the absolute center of the column, the columns do not have great broad heads top and bottom. What's the answer? An appalling number of buildings have collapsed by reason of the weakness of the columns which the framers of this report would continue to foist on the innocent public.

MR. E. G. PERROT.—Mr. President, I have been very much interested in the result of these tests as a means of determining in a practical way the working values which we can use for hooped columns. I have devised a formula which we have been using since 1906, based on tests made at the Watertown Arsenal and Considere's test, which, I think, is a little different from the accepted formulas now used by engineers. We have built a number of buildings in the past ten years, using the formula, and have had very successful results, furthermore we have over- Mr. Perrot.

Mr. Perrot. loaded these columns, in other words, have put an additional story on one building which was designed for six stories originally, making seven stories, using hooped columns 22 in. in diameter, and high values on the concrete with 10 per cent of vertical reinforcement.

There is no new principle involved in this formula, only the theoretical reasoning of the resolution of forces and the application of the well-understood physical properties of concrete and other building materials.

I am indebted to Prof. John Goodman's (of Leeds University) book entitled, "Mechanics Applied to Engineering," for the suggestion which forms the basis of the formula. Regarding

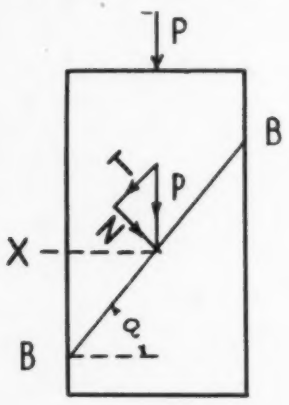

FIG. 1.

the behavior of material under compression, he clearly demonstrates, both mathematically and from experiments, how brittle materials in compression fail by shearing of one-half of the specimen over the other. By theoretical reasoning, he arrives at the manner in which a Portland Cement cube should fail, namely, as two pyramids with their apexes in the middle. As the material fails by shear, that is to say, bursts under compression before it is actually crushed, he figured that the vertical load applied to the specimen can be resolved into two components to the plane of the shear—one normal and the other tangential. By assuming a plane of shear at an angle of 60 deg. which has been found by experiment and corroborated by shear failures in columns at

the Edison fire, we can determine the horizontal component of the shearing force, which, for our purposes in hooped columns, is one which must be resisted by the hooping. To ascertain the shearing stress we may reason as follows:— Mr. Perrot.

Let the sectional area of the prism $= A$, then the unit stress section $S = \frac{P}{A}$ and the stress on the oblique section BB making an angle a with the cross-section, may be found thus: resolve P into components normal $N = P \cos a$ and tangential $T = P \sin a$.

The area of the oblique section $BB = A_1 = \frac{A}{\cos a}$,

the normal stress $= \frac{N}{A_1} = \frac{P \cos a}{\frac{A}{\cos a}} = \frac{P \cos^2 a}{A}$,

tangential or shearing stress $\frac{T}{A_1} = \frac{P \sin a}{\frac{A}{\cos a}} = \frac{P \cos a \sin a}{A}$

$= S \cos a \sin a.$

Using 60 deg. as the shearing angle of concrete, the shearing stress at this angle would be $S \times .5 \times .86 = .43S$. But as the column is hooped the horizontal component of this shearing stress will have to be resisted by the hoops, this is, therefore, the bursting pressure which we will call X and is found to be cos a $\times$ the shearing stress, or $X = .5 \times .43S$ when $A = 60$ deg.

Following is the formula:

$$S = .215 \times C \times \frac{d}{2} \times U$$

in which

S = Stress in one hoop in lb. per sq. in.

U = Spacing of hoops. (This should never be more than 3 in. and usually $\frac{1}{7}$ to $\frac{1}{8}$ the diameter of the column.)

d = Diameter of column in inches to center of hoops.

c = The reduced unit stress on the concrete.

In using the formula, we first find the diameter of the columns within the hoops by dividing the load to be carried by

Mr. Perrot. 1000 lb. Assuming the total load on a column of 421,000 lb. we have:

$$421{,}000 \div 1000 = 421 \text{ sq. in. required.}$$

The nearest circle to this in area is 23 in. in diameter = 415 sq. in.

In order to find the bursting tendency, which must be resisted by the hooping, we deduct from the total load on the column the total tangential shearing strength of the concrete figured at 215 lb. per sq. in. The value of the 215 lb. for the shearing strength of the concrete is based on the assumption that if 500 lb. is the nominal safe load on the concrete in direct compression, the shearing strength for such concrete, based on the above reasoning, should be 43 per cent of 500, or 215 lb. per sq. in. The area of this oblique section is equal to the area of an ellipse whose long diameter is twice the short, or for a 23 in. column the area of the oblique section would be $23 \times 46 \times .78 = 825$ sq. in., which multiplied by $215 = 177{,}375$ lb.

The total load deduct $= 421{,}000$ lb.

$177{,}375$

$243{,}625$ lb. the tangential force of which will produce a bursting effect.

The reduced unit stress c on the column from the load of

$$243{,}625 \text{ lb.} = \frac{243{,}625}{415} = 587 \text{ lb. per sq. in.}$$

The stress in a hoop

$$S = .215 \times 587 \times \frac{23}{2} \times 2.5 \text{ in.}$$

This gives 3621 lb. as the stress in a hoop. Divide this by 16,000 lb., the unit tensile strength of the steel, we have .22 sq. in. cross-section necessary a $\frac{9}{16}$ round rod = .248 sq. in. and this should be used at a $2\frac{1}{2}$ in. pitch or spacing.

There should be eight $\frac{3}{4}$ or $\frac{7}{8}$ in. vertical rods inside the hooping to act as spacing rods for the hoops, which should be wired to them. A helix makes the best hooped column, as there are no ends except at top and bottom. The longitudinal rods should run 3 ft. into the column above. No load is assumed to be carried by them. Sometimes it happens that additional rods figured

to take some of the load are found necessary. It is better then to place these at the center of the column. These can be figured at as high as 10,000 lb. per sq. in. Of course, a flat bearing plate must be provided at the bottom and the ends of the column rods either sawed square or otherwise given a true bearing. If these rods extend through more than one story, they can be joined one ft. above the floor line and pipe sleeves placed about the joint to keep the rods in alignment. Of course, these ends must butt true. Mr. Perrot.

The mixture of concrete should be 1 of cement, 1 of sand and 2 of stone of a size to pass through a $\frac{3}{4}$-in. ring. The columns should have 2 to 3 in. of concrete outside the hooping for fireproofing.

Now what I would like the Committee to consider is this: Whether it is possible from tests, if it has the data, to find out what the stress would be in the hoops at the time of their fracture. I present this with the view of having it look into it because if there is something more rational to be gotten up, I think it is time for this Committee to take up the hooped columns, as they have the beams. We all agree that the parabolic formula for the compression of a deep beam is theoretically more correct than the straight line formula, but we do not use it because it is clumsy.

MR. A. E. LINDAU.—In reply to Mr. Godfrey's criticism that these columns are not columns but that they are piers, it might be worth while to remark that considerable thought was given by the Committee to the size of the test specimen and the details of its design; and it was decided that a very large column, first of all, should be made, one that was much larger than any which had been tested heretofore, and if possible, to get such a test section or test piece as would represent an average column as used in a building, and the Committee feels that a 20-in. column 12 ft. long, 12 ft. being the ordinary story height, is an average column. The column in the top story of a building might be 12 in. and the bottom probably 36 in., but the 20-in. column is the average building column, and is the kind we wanted to test. Mr. Lindau.

In regard to the increased strength and the effect of various types of longitudinal and spiral reinforcement, that is illustrated on page 429, for example: the increase in strength for increasing percentages of longitudinal reinforcement with six percentages of spiral reinforcement.

Mr. Lindau. Fig. 12, on page 430, shows the increased strength varying with the ratio spiral reinforcement with a fixed percentage of longitudinal reinforcement. These diagrams represent results on these tests. There is no argument about it. We are dealing with established facts.

With regard to Mr. Perrot's discussion on the tests of stress in spirals, it would be impossible to make a reply offhand. It would appear to the speaker that the condition to which he is calling attention, more nearly applies after the concrete has failed, after the shearing plane has developed, and that it would not apply during the working conditions of the column; that is, the swelling of the concrete, which takes place under the load, is represented by the diagrams which are in our test reports, but little information is available regarding stresses in spirals after the concrete has failed. An idea of the stresses, in the spirals at those loads, can be found from an examination of the tests. Just what formula will represent these conditions, and whether any formula can be made which would represent conditions continuously from the beginning of the working loads, I am not in a position to say.

Mr. Mensch. MR. L. J. MENSCH.—Mr. Perrot's discussion is an explanation of Considere's theory. To my knowledge, it has never been published in the United States, although Mr. Considere mentioned it in his first publication of his tests in 1902, but he did not explain it. He simply showed the diagram while Mr. Perrot explained where the pressure on the hoops comes from and I would suggest that Mr. Perrot revise his discussion a little and that it be incorporated in this report, because it will explain the action of hooped columns.

Mr. Talbot. PROFESSOR TALBOT.—With reference to the matter brought up by Mr. Perrot I was, of course, unable to follow it from the diagram on the screen, but I believe the Committee will be glad to have his discussion and to make comparison with the measurements which have been made.

So far as I have been able to find from an examination of analysis based upon shear, there is nothing to indicate that the failure is due to shear. The stresses do vary somewhat from top to bottom, and around the column, but there are other gradations in the amount of the stress and of the general bulging. Data to

which he refers, however, are available for calculation. There was a determination of the stress-strain relation of the steel, so that if the steel stretched a certain amount the corresponding value of the tensile stress in the reinforcement may be calculated. And it is noted in the report that the additional stress, the additional load put upon the column beyond that which it presumably would have held if it had not been reinforced with the spiral, was almost directly proportional to the amount of stress developed in the spiral itself up to a point where perhaps 85 per cent of the full strength of the steel was developed. I think that is an important feature of these tests. Mr. Talbot.

Further, in these tests and in a number of tests which have been carried on in the University of Illinois within the last few years, we have data on this relation of stress in the spiral. If we consider that this is a bursting action and that the load which is producing this is the excess of load over that which presumably would be carried by an unhooped concrete column, then consider that the effect on the spiral is the same as though produced by internal hydraulic pressure, we can find the ratio of this hydraulic pressure representing the lateral pressure to the excess of longitudinal pressure referred to above. The values for the columns tested indicate that this lateral pressure, which is dependent, you see, upon friction of the particles or something connected with the resistance of motion of one particle over another or perhaps distortion of particles, has a ratio to the longitudinal stress corresponding to the excess of load above referred to, which runs from 25 to 35 per cent—these values, presumably, depending upon the nature of the aggregate as well as the mix of the concrete.

MR. PERROT.—I am very thankful to Professor Talbot for the remarks he has made on this subject. I would like to call his attention, if he does not already know of it, to the works of Professor Goodman, because he shows tests which he made on cast-iron cylinders the height of which were greater than the diameter; and they sheared off after they had bulged. They bulged and then sheared off at approximately an angle of 60 deg., showing that there was shearing action when the ultimate strength of the cylinder was reached. Therefore, I work on the theory, in designing a building that what any architect or engineer must know is Mr. Perrot.

Mr. Perrot. the ultimate strength of the structure; because all of our building regulations are based on loads being one-quarter of the ultimate strength. This is not a mathematical deduction, however. The general assumption in most building laws is that it will take four times the load to collapse a building, not to have it bend a little and show cracks. Consequently if we can arrive at some formula as Mr. Mensch has, in terms of ultimate destruction of the column, and then work backward from that to a safe working load on the column, I think we are absolutely safe provided we also carry our deduction far enough to know when the critical moment occurs.

These tests which were made under the direction of the committee are very full and give the critical moment, but as far as the building regulations go, we are supposed to have our building four times stronger than the ultimate breaking load, so let us make our hoops strong enough to resist the tangential force of the shear. If we do that we will be on the safe side, and you will find that the column designed as I have proposed, is a commercial column, although it has more steel in the hoops than what the test column shows for carrying a similiar load.

Mr. Mensch. MR. MENSCH.—I have before me the tests of the French Commission and I find that some of the reinforced columns 8 in. in diameter and about 18 to 4 in. high reinforced by 1 per cent and 1.3 per cent of hooping. The diameter of the hooping was about one inch less than the diameter of the column. The stresses at the ultimate load were from 5,000 to 7,000 lbs. per sq. in., hence they agree very well with our own tests. It is to be remembered that the relation of length to the diameter in the French tests is nearly 20. They show that a hooped column is very safe. I wish to add that hooped columns are patented and we are going ahead with the unauthorized use of the hooped columns and we don't know when a high royalty will be demanded.

Mr. Tubesing. MR. TUBESING.—In relation to what Mr. Mensch has just said, I designed years ago, in Milwaukee, a seven-story structure, and I asked if the committee would kindly include one column, at least, in their tests. This was an octagon column with octagon hoops and figured by Considere's formula, that the amount of extra concrete which I got in compression for the same sized column—that is, occupying the same floor space—gave me a more

economical column. This building has been up since 1907 and there has not been any spalling of any kind that I can see. Those who were selling spiral reinforcement told me that my octagon hoops would take a circular shape, which, as we all know, could only take place as result of failure. I think that in the next series of tests which are made, one or two of these octagon-hooped columns should be included. I believe it will give some information which might be valuable, because I have never seen a test made on this type of column. I have obtained more for the amount of money spent than I could otherwise, because these columns were very easily formed, as may be seen in Fig. 2. It required a square box column with the bevel boards put on each inside corner of the column forms instead of the usual chamber strip. Will the Committee please consider this? Mr. Tubesing.

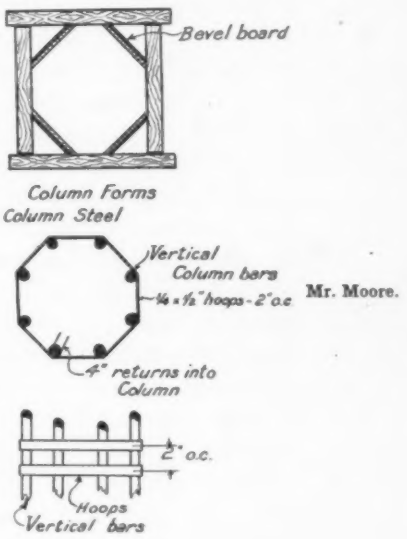

FIG. 2.

MR. E. J. MOORE.—While it is considered by the Committee best to regard the unit ultimate load carried by the column for the cross-section, it seems to me that designers will probably be interested to know what the unit ultimate load carried by the columns was within the section of the diameter of the spiral. I would be very glad to furnish these figures, and suggest that they be incorporated as part of the discussion of this report. Mr. Moore.

MR. T. L. CONDRON.—I feel as if the results of the tests speak for themselves. We are dealing with facts here rather than with our opinions, and I am willing to take those facts which have been presented and study them carefully either to confirm the opinions I have had or have my opinions modified. If the spiral hooping is to withstand the stress which we think it should be made to withstand, it should be in the form of a circle. They are more cheaply made in that form than in any other. The column which is fireproof is presumably designed to withstand Mr. Condron.

Mr. Condron. the action of fire. We have recently demonstrated that the action of fire tends to make a column cylindrical during the conflagration. I would, therefore, suggest to Mr. Tubesing that he cheapen his columns still further by building them round, and also materially reduce the cost of his spirals by making them circular instead of octagonal.

Mr. Tubesing. MR. TUBESING.—In reply to Mr. Condron, I wish to say that I have kept costs of both of these types of column construction. I have used round columns, mushroom shapes and every other type. The result obtained was from an actual cost study and showed that the octagon-hooped column is cheaper than the round column. If we are going to pay a royalty then it concerns the contractor and the engineer.

Mr. Mensch. MR. MENSCH.—In regard to the square-hooped columns, I wish to state that the patent office in Germany declared the patent of Abramahoff on square hooping void, and an infraction of Considere's patent. Wayss & Frietag, the owners of Considere's patent in Germany, made tests on square-hooped columns on a large scale and found that square hooping, consisting of wires less than $\frac{1}{4}$ in. thick, are just as effective as round hooping. There are a few cases where these columns may be used to advantage. I am just now figuring on a building where there are arches very highly stressed and the architectural requirements are that the section of the arch must be a rectangle. Using round hooping, the core area would be considerably smaller than the core area of a rectangle. It is certainly surprising that square hooping is just as effective (provided thin wires are used) as round hooping, but reliable tests prove it to be the case.

Mr. Condron. MR. CONDRON.—If the Considere column or the hooped column is patented in this country, there is no engineer or anyone else who should refuse to use a good thing because it is patented. It is perfectly feasible, if there is a patent covering the hooped column, to arrange with the patentee for a license. It is done in all things except in structural engineering, where it is customary to beat the patent rather than pay the royalty. But in mechanical engineering, in plumbing and in many other lines, the inventor is encouraged to get out a good thing, and if he does he has the use of it and gets a royalty. If there is a patent on the hooped column which can be reasonably established, it should be treated

the same way as the Gray patent on the steel column. Many of us use that column in structural steel work and arrange that the royalty be paid to the patentee. It is not a serious burden on anybody and if it is a patent, I think it should be recognized. I think I, for one, and many others would not hesitate to use it because it is patented, but would recommend to our clients to use it and recommend that the royalty for the use of it be paid. Mr. Condron.

MR. MENSCH.—Mr. Considere wrote me and asked me to handle the patent for him in a certain district. I think he wanted $2.00 or $4.00 a ton royalty. Mr. Mensch.

MR. CONDRON.—Weight of column or weight of spiral? Mr. Condron.

MR. MENSCH.—Weight of spiral. Mr. Mensch.

MR. CONDRON.—That would be a mere bagatelle. Mr. Condron.

MR. LINDAU.—If those who have copies of this report, after they have looked over and digested it, will be kind enough to send suggestions for further investigations on columns to the office of the Institute, it will be very much appreciated by the Committee. Mr. Lindau.

PRESIDENT HUMPHREY.—The discussion of this report can be carried on in the JOURNAL after you have had an opportunity for more carefully studying the tests. The President.